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ERRATA

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Page 902, Line 10 from bottom, for "heard" read "saw".

Page 912, Line 5 from top, for "involved" read "invoked".

Page 914, Line 16 from bottom, for "such" read "public utilities".

Page 1071, Fig. 7, the scale of vapor pressure is shown in inches; it should be tenths of inches.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

TRANSACTIONS

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Paper No. 1335

HISTORY OF LITTLE ROCK JUNCTION RAILWAY BRIDGE, OF THE ST. LOUIS, IRON MOUNTAIN AND SOUTHERN RAILWAY COMPANY, OVER THE ARKANSAS RIVER AT LITTLE ROCK, ARKANSAS, 1883-1914*

By C. E. Smith, M. Am. Soc. C. E.

With Discussion by Messrs. Henry H. Quimby, M. L. Byers, Lee Highley, Theodore Belzner, Robert H. P. Ford, C. D. Purdon, F. G. Jonah, and C. E. Smith.

Synopsis.

Object.—**This paper was written for the purpose of informing the Engineering Profession of the troubles encountered at the Little Rock Junction Railway Bridge, due to the defective construction of the piers, in order that such defective construction may be avoided in the future; its purpose is also to point out the methods suggested and used to accomplish the correction of the defects, in order to guide the Profession in the correction of similar defects which may cause trouble in future in similar piers.**

Digest.—**The bridge consists of three 253 ft. 4-in., through truss spans, one 127 ft. 11-in., through truss span, and one 352 ft. 10-in., through truss swing draw-span, all single-track, resting on masonry**

* Presented at the meeting of December 16th, 1914.

piers and one abutment. Four of the piers are supported on pneumatic caissons and timber cribs. The pneumatic caissons are of ordinary timber construction, and rest on bed-rock, approximately 45 ft. below low-water line.

The timber cribs over the pneumatic caissons, extending up to low water, consist of 12 by 12-in. timbers, 3 ft. from center to center, sheeted outside with 3-in. planks. The spaces between the timbers were supposed to have been filled with rip-rap, but only a small quantity was placed, the greater number of the cavities having been filled with sand discharged from the caissons.

The caissons were poorly located, and when the working chambers were finally sealed, the tops of the timber cribs were considerably out of place. As the steel superstructure was on the ground at the time of the commencement of the masonry work, the masonry piers were located on top of the cribs in such a way that the coping courses were the proper distances apart to receive the steelwork, which made it necessary to place the piers at one side or the other of the timber cribs and use different batters.

The bed of the Arkansas River is composed of fine sand and silt, which is scoured to great depths during floods, especially around obstructions such as piers. The scouring of the river around these piers caused the sand to run out of the cribs through the spaces between the sheeting planks, and the timbers, being deprived of the support of the filling material, settled under the weight of the bridge and traffic.

As the masonry piers were not located concentrically with the cribs, the timbers compressed unequally and caused the piers to lean, their tops moving in various directions. The piers were a constant source of trouble and expense from the date of construction in 1883 until recently when they were reconstructed.

The reconstruction consisted in placing annular pneumatic caissons around the old piers, filling the spaces in the old cribs with concrete, enlarging the footing by filling in with concrete over the new annular caissons, and encasing the old piers in reinforced concrete following symmetrical lines, and enlarged for a double-track superstructure in the future.

The excessive scour of the Arkansas River renders practically impossible the maintenance of falsework for long periods, and swift run-outs have been experienced in all months. As the hazard attending

the maintenance is very great, the work was carried out without falsework.

The paper also describes in detail the various investigations to which this bridge was subjected by engineers, and the various unsuccessful attempts, made at great expense, to correct the trouble in advance of the final reconstruction.

Conclusions.—Timber cribs extending from pneumatic caissons to low-water level are entirely unsuitable for bridge pier foundations unless they are of solid timber construction, or, preferably, if the spaces between the timbers are carefully packed full of rock and thoroughly grouted, or, best of all, if the spaces are filled carefully with concrete and well rammed so as to fill all voids.

When defects are discovered in bridge piers, and trouble results, correction should be applied at the seat of the trouble; expenditures for temporary expedients are a waste of money. The continuation of the trouble without adequate correction involves constant hazard, which can be avoided by efficient engineering talent.

Descriptive.

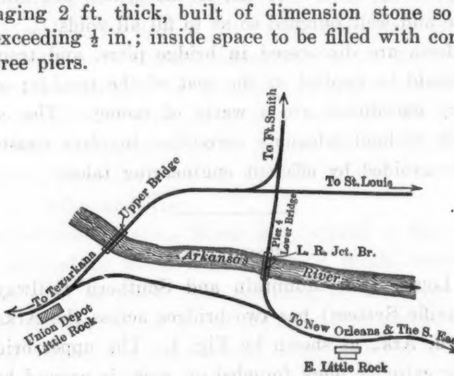
The St. Louis, Iron Mountain and Southern Railway Company (Missouri Pacific System) has two bridges across the Arkansas River at Little Rock, Ark., as shown by Fig. 1. The upper bridge, resting on pneumatic cylinder piers founded on rock, is crossed by main-line freight and passenger trains from points northeast of Little Rock to Texas and the Southwest. The lower bridge, founded on masonry piers, timber cribs, and pneumatic caissons on rock, is used by freight trains only from northeastern points to Louisiana and Southeastern Arkansas. Plate I gives an elevation of this structure, commonly known as the Little Rock Junction Bridge. It consists of one 352 ft. 10-in. swing draw-span, three 253 ft. 4-in. simple truss spans, and one 127 ft. 11-in. simple truss span and trestle approach, all single-track, built in 1883. Fig. 2 is a view of the structure. The south abutment is of masonry built on the rock that outcrops on the south bank. The pivot pier, commonly known as Pier 2, and Piers 3, 4, and 5, are rock-faced, concrete-filled, masonry piers, about 45 ft. high, resting on filled timber cribs and pneumatic caissons about 40 ft. high, the masonry of the draw-pier being annular, with

a well down to the crib. Piers 6 and 7, are rock-faced, concrete-filled, masonry piers built on piles. The only specifications that can be found for the piers are as follows:

"The piers to rest on pneumatic caissons sunk to rock filled with concrete, timber cribbing to reach from roof of caissons to 4 ft. below low-water mark.

"This crib to be drift-bolted and planked on outside and filled with sand and stone; on this crib the masonry to be started and built up to grade line.

"Piers to be 6 ft. 6 in. wide under coping and 20 ft. long, with semicircular ends, with a batter of ½ in. per ft. on sides and up-stream ends, but no batter on down-stream ends; piers to consist of solid walls, averaging 2 ft. thick, built of dimension stone so as to make joints not exceeding ½ in.; inside space to be filled with concrete. This refers to three piers.



LOCATION OF LITTLE ROCK JUNCTION BRIDGE, LITTLE ROCK, ARK.

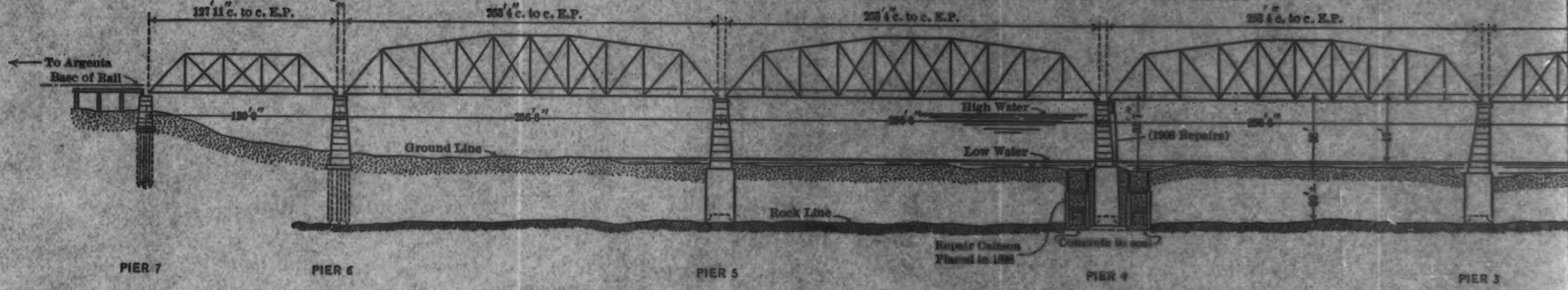
FIG. 1.

"Draw-pier to be 30 ft. in diameter under coping, with 8-ft. wall; center to be left open; pier to rest on caissons sunk to rock and filled with concrete with crib as before.

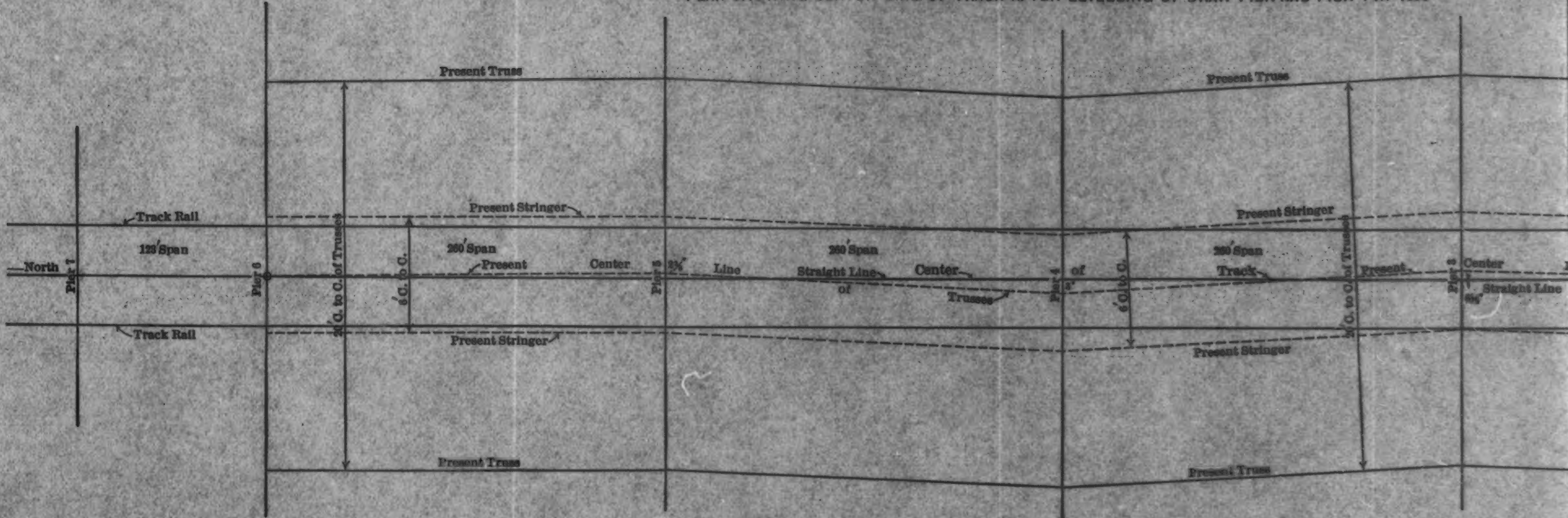
"Fifth pier on east bank to rest on piles sawed off and capped below low-water mark; to be 6 ft. 6 in. wide under coping and 20 ft. long, with semicircular ends, to have a batter of ½ in. per ft. on sides, but no batter on ends; in other respects to be the same as Piers 1, 2, and 3. All work to be done in a thorough workmanlike manner and to the satisfaction of the General Manager of the Little Rock Junction Railway.

"The draw protection and draw-pier to be completed by the first day of June, 1884, and all the work embraced in this contract to be completed by the first day of September, 1884, unless some disaster by flood or by other uncontrollable cause shall prevent.

SIDE ELEVATION OF LITTLE ROCK JCT. BRIDGE



LITTLE ROCK JUNCTION BRIDGE
PLAN SHOWING CENTER LINE OF TRACK AFTER LEVELLING UP DRAW PIER AND PIER 4 IN 1899



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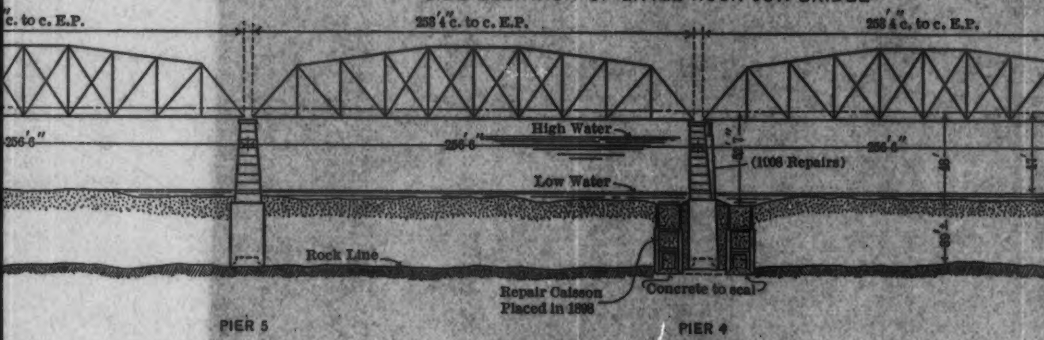

LITTLE ROCK JUNCTION BRIDGE
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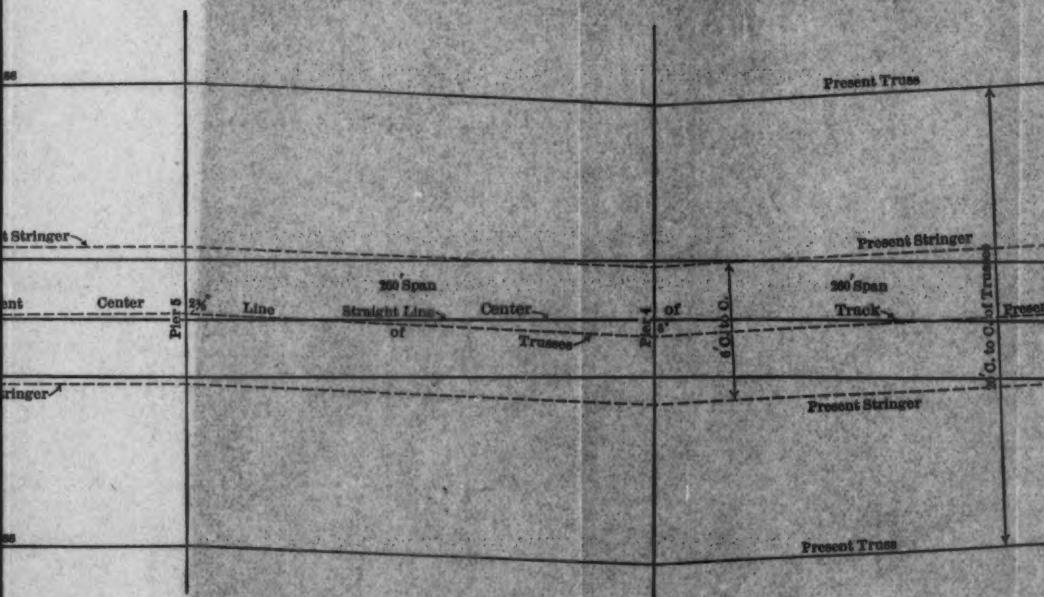

PLATE I.
TRANS. AM. SOC. CIV. ENGRS.
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SMITH ON
REPAIRING BRIDGE PIERS.

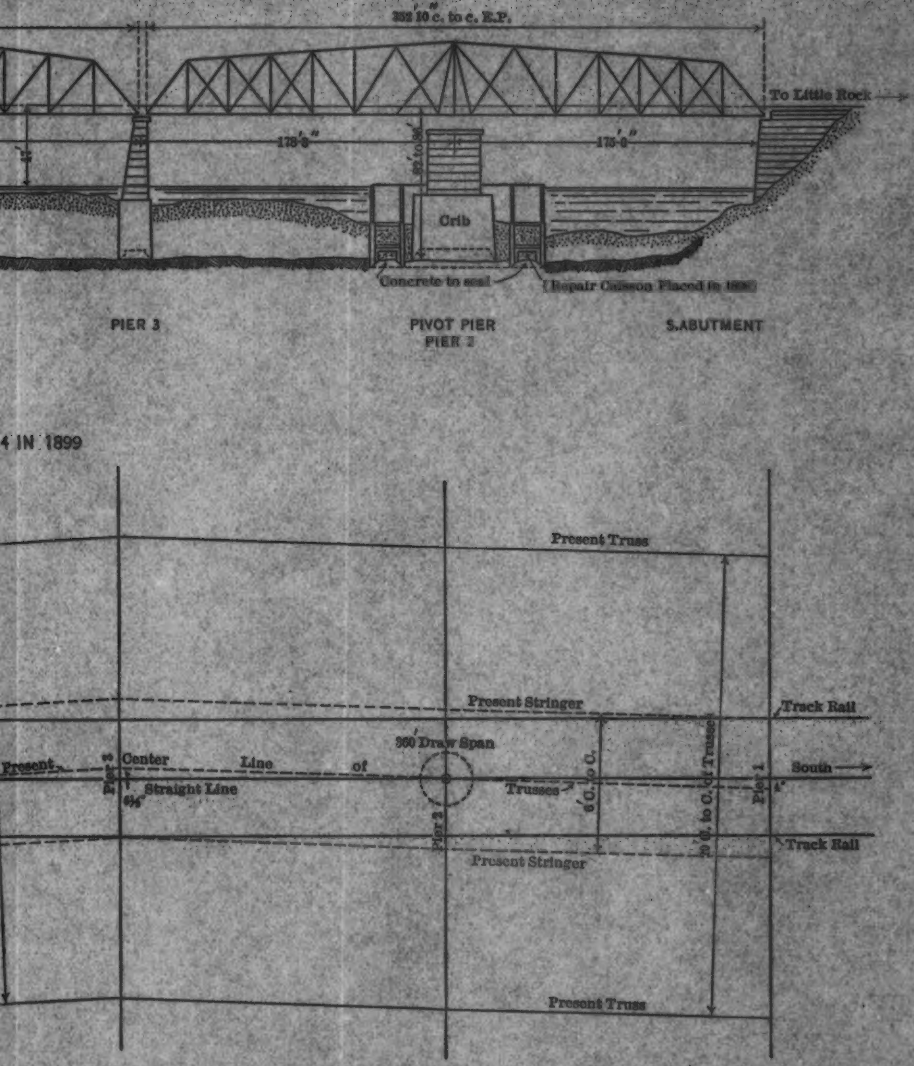


FIG. 2.—LITTLE ROCK JUNCTION BRIDGE, LITTLE ROCK, ARK.

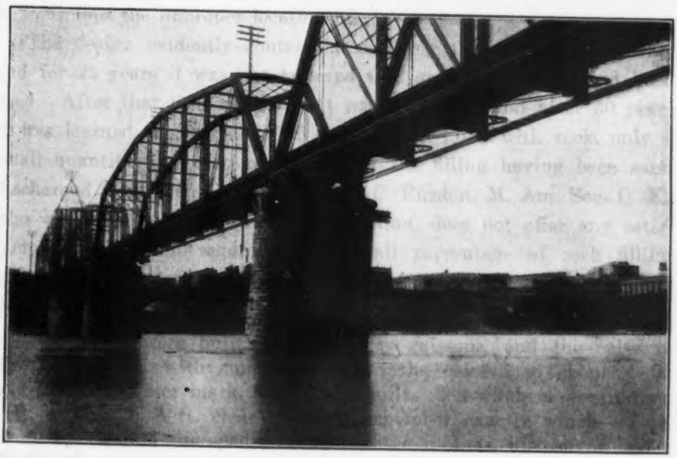


FIG. 3.—PIER 4 AFTER APPLICATION OF YOKES.

"Modifications may be made in these plans and specifications upon the request in writing by the General Manager of the Little Rock Junction Railway, and these modifications shall be complied with by the Contractors, but if the modifications increase the cost of the work, such increased cost shall be paid to the contractors."

The spans were so well designed and constructed that they are now safely carrying the present-day heavy engines and trains, and will continue to do so for many years. The design and construction of the south abutment and of Piers 6 and 7 were good and well executed. The design of the four pneumatic piers, although not in accordance with the best present-day practice, was adequate, but the construction was so faulty that trouble was experienced with them from the first, and the efforts that have been made during a period of 30 years to correct the defects resulted in a sequence of events that partook of the nature of a farce comedy in the face of impending disaster, which latter was narrowly averted.

There was nothing unusual or defective about the design or construction of the caissons, but they were very poorly located and carelessly controlled during sinking, resulting in their having been founded considerably out of place—from 2 to 3 ft. in one or two cases. The timber cribs, extending vertically upward from the caissons, reflected at their tops the improper location of the caissons.

The design evidently contemplated filling the cribs with rip-rap, and for 25 years it was not believed that any other filling had been used. After that time, however, it was suspected, and after 30 years it was learned, that, instead of having been filled with rock, only a small quantity had been used, most of the filling having been sand discharged from the caisson. C. de la C. Purdon, M. Am. Soc. C. E., who was inspector during the construction, does not offer any satisfactory explanation regarding the small percentage of rock filling and large percentage of sand filling, a letter from him in reference thereto reading as follows:

"The piers were built with ordinary caissons, and the caissons themselves filled with concrete. Above the caisson to about 4 ft. below the low-water mark, a crib was built. The timbers were either 3 ft. centers or 3 ft. clear—I don't remember exactly which—being all 12 by 12, and the spaces between these timbers, instead of being filled with concrete as is customary, were filled with rip-rap and then sand washed in. I objected to this strongly at the time, but as I

did not design the piers, this work being done by the late Mr. T. E. Sickels, I was not responsible for them. I told Mr. Wood at the time that I was satisfied the timbers would eventually crush and put the piers out of shape, which it seems occurred."

This does not entirely explain the trouble. Had the cribs been properly filled, the rip-rap filling every part, and then with sand washed into the interstices of the rip-rap, little, if any, settlement would have taken place. Very little rip-rap was placed, the greater part of the filling being sand, and, when this leaked out, the small quantity of rip-rap settled through the cribs, leaving the timbers to carry all the load. No satisfactory explanation has been found for filling the cribs with sand instead of stone.

The incorrect location of the caissons and cribs was discovered before starting the masonry, and as the spans were on the ground, the errors in location were corrected partly by placing the masonry piers to one side or the other of the cribs, partly in the batter of the piers, and partly in the placing of the bed-plates on the pier tops.

Pier 4 was built near the north edge of the crib and given equal batter on the two sides, and Pier 3, the north rest pier of the draw-span, was built near the center of the crib; but the north face was given a batter of 1½ in. per ft., and the south face was built plumb; the draw-span barely got a bearing on Pier 3, and the next fixed span reached well over on the pier.

Fig. 4 shows the location of the footing courses of Piers 3 and 4 with reference to the tops of the cribs, the cross-section of Pier 4, and the location of the intermediate span from Pier 3 to Pier 4.

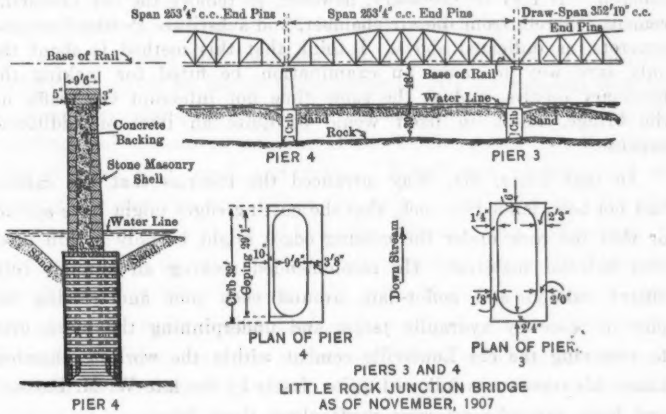
Early Trouble.

The early record of the trouble is not clear, but it appears that, immediately after the completion of the bridge, the pivot pier under the draw-span (Pier 2) and Pier 4 began to settle and lean. As the bed of the Arkansas River is composed of fine sand which scours and shifts greatly during floods, it was thought that the settling was due to scour, the opinion immediately being formed that the cutting edges had not been founded on rock. Consequently, large quantities of rip-rap were unloaded around the piers, only to be washed down stream in following floods and requiring replacement. In addition, from time to time as necessity arose, the spans were

shifted back and forth, to keep their bearings on the piers, the tops of which had been made so small that very little variation could be permitted. The movement of the pivot pier was quite pronounced, and necessitated frequent leveling and adjustment of the draw-span, at great expense.

The matter was the subject of continual correspondence between the various officers, and, in reply to inquiry as to the best manner of strengthening the piers, Chief Engineer James W. Way, on December 6th, 1897, recommended as follows:

"Replying to your inquiry as to best method of proceeding with the work of strengthening the foundations of the pivot pier and pier 4 (numbering from the south) of the Little Rock Junction Bridge across the Arkansas River at Little Rock.



PIERS 3 AND 4
LITTLE ROCK JUNCTION BRIDGE
AS OF NOVEMBER, 1907

FIG. 4.

"One theory which may be presented for consideration is that the caissons are not properly landed on bed-rock, or possibly they may be faulty in construction and have spread at the cutting edges or on the corners. Again, it may be that what was termed bed-rock and believed to be such when the caissons were put in place, was but a thin stratum of shale overlying softer material; and that the respective weights of the piers, with their loads have caused this stratum to settle.

"This can readily be determined, if indeed it has not already been done, by making borings three or more feet below the elevation of the cutting edges of the caissons. Should the borings show solid

rock, then the natural inference is that the caissons are imperfectly constructed. I would therefore suggest that a careful examination of the bed of the river be made and the character of the obstructions ascertained. Boring should then be driven to disclose the nature of the rock on bottom. This should be done very carefully; and finally the caisson or crib should be inspected to ascertain condition of timbers, etc.

"A double steel caisson of sufficient diameter and with four or five compartments could then be sunk to bed-rock, using compressed air to remove the débris between the two cylinders. When this is done the space between the inside cylinder of the steel caisson and the wooden crib can be pumped and cleaned out to bed-rock; when an inspection will disclose the best method* of correcting the trouble.

"It is possible that by the use of hydraulic jacks the pier can be raised to place and the air chamber filled with Portland cement concrete. It may be necessary, however, to remove the old Louisville cement concrete from the air chamber, and substitute Portland cement concrete, as indicated above. I think that this method is about the only sure way to make an examination, be fitted for making the necessary repairs, and at the same time not interrupt the traffic on the bridge, which in itself would be quite an item of additional expense."

In that letter, Mr. Way advanced the theories that the caisson had not been landed on rock, that the cutting edges might have spread, or that the rock under the cutting edges might be only a thin layer over inferior material. He recommended sinking an annular (circular) caisson and coffer-dam around each pier and raising the pier to place by hydraulic jacks, and underpinning the piers, even to removing the old Louisville cement within the working chamber. Later, his report was followed quite closely by the late W. M. Patton,* and large expenditures were made along those lines.

After the Railway Company's forces had handled the problem for 15 years, Mr. Patton was called in as Consulting Engineer. He studied the history of the bridge and, after making borings through the timber crib and caisson under the pivot pier, within the well inside the pier, and examining the exposed portions of the crib by divers, came to the conclusion that the caissons were not founded on rock, and gave the Railway Company a full and very plausible report and recommendation, which, on account of its unusual nature, is reprinted in full in Appendix A.

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* Author of "Treatise on Foundations."

Patton's Report.

In brief, Mr. Patton concluded that one corner of the cutting edge of each pier rested on rock and the remainder on inferior material, which condition, together with the eccentricity of loading, caused the settlement; that greater scour at one corner than at the others caused the greatest settlement at the corner of greatest scour. He suggested pumping grout into the underlying sand to convert it into concrete; removing the inferior material under the caisson and replacing it with concrete, and, to accomplish the latter, discussed freezing the sand to the river bed to form a frozen sand coffer-dam. He also suggested an ordinary coffer-dam, which he dismissed on account of the difficulty of getting it tight in that material and at that depth; proposed the use of a pneumatic caisson with the old crib and caisson as one of its walls, which latter he concluded to be very risky; and suggested an entirely independent annular coffer-dam, which he finally recommended. His diver found that the timbers were sound, but that some damage had occurred to the crib sheeting and timbers on account of the settling; and that the filling was of small rock, sand, and gravel. Mr. Patton recommended replacing the sheeting and filling the spaces with concrete. He also recommended the sinking of annular caissons and coffer-dams to rock, supporting the span on falsework, removing the material between the old and new piers, putting jacks under the cutting edges to right the piers, and underpinning them with concrete. As an afterthought, he suggested that the rock, under the cutting edges at the sides of the pier which had not settled, might be cut out, and those edges lowered to make the piers assume perpendicular positions. He went so far as to say that from 30 to 35 hydraulic jacks, with main cylinders 3½ in. in diameter and plungers ¾ in. in diameter, would raise 30 tons each. It is not known how he proposed to work the jacks when under the cutting edge, as he recommended letting the annular caisson flood while jacking so as to reduce the weight to be lifted. His estimated cost was $40 000 for the coffer-dam around the pivot pier, with $4 000 for raising and leveling it, and $34 000 and $3 500, respectively, for Pier 4, a total of $81 500.

It is puzzling that Mr. Patton did not determine by simple calculation that, if Pier 4 could be righted by such a method as he suggested,

the top would be in such a position that the ends of the spans supported by it would not fit the top of the pier in the new location, as it would have been moved over 3 or 4 ft. It also appears that he ignored the statements of the contractor and resident engineer who built the piers that they were surely founded on rock. Later, the sinking of the coffer-dam around Pier 4 disclosed the fact that the rock slopes in a direction opposite to the slope on which he based his theory, a fact that could easily have been determined by running jet pipes down to rock at several places around the pier.

Annular Coffer-Dams Sunk in 1898 and 1899.

Mr. Patton's report was accepted, and bids were requested on his plans and specifications, with the following results:

Engineering-Contracting Company. (Charles Sooysmith, M. Am. Soc. C. E.):

Pivot pier, $38 332. $1 500 per ft. below 40 ft. below low water.

Pier 4, $36 010. $1 200 per ft. below 45 ft. below low water.

$18 per cu. yd. for work below the cutting edge of both piers.

Missouri Valley Bridge and Iron Company. (The late A. J. Tullock, M. Am. Soc. C. E.):

Pivot pier, $31 600. $800 per ft. below 40 ft.

Pier 4, $27 950. $750 per ft. below 45 ft.

Removal of material between new coffer-dam and old pier and all other work, cost plus 10 per cent.

McGee-Kahmann and Company, Kansas City:

Both piers, $116 000.

Excavation of material between, $1 per cu. yd.

Three companies made propositions for building new piers, as follows:

McGee-Kahmann and Company proposed to build four new piers, 50 ft. north of the present piers, launch the draw-span 50 ft. north, and construct a new span 50 ft. shorter between Piers 5 and 6 and a deck plate-girder at the south end, for $99 000. They also made an alternate bid to build a new draw-span, 80 ft. longer than the old one, reaching from the south abutment to the new piers, renewing Piers 2, 3, and 4, farther north, and to build a new span between Piers 4 and 5, for $125 000. The latter plan would have left in the defective Pier 5.

The Missouri Valley Bridge and Iron Company proposed to build an entirely new set of piers, according to the best modern practice, for $125 000.

All propositions for new piers were rejected, and the bid of the Missouri Valley Bridge and Iron Company for going ahead on Mr. Patton's recommendation was accepted. Mr. A. J. Tullock, then Proprietor of the Missouri Valley Bridge and Iron Company, had made a careful investigation before bidding; the correspondence accompanying his bid pointed out the weaknesses of Mr. Patton's scheme, and made certain recommendations. On account of the value of Mr. Tullock's conclusions, his letters of August 17th, 18th, and 19th, 1898, to Mr. E. Fisher, then Engineer of Bridges and Buildings, of the Railway Company, are given in full in Appendix B.

As a result of his study, Mr. Tullock raised a doubt as to the correctness of the theories formerly advanced regarding the trouble, and attributed it to the crushing of the crib timbers. He pointed out the impracticability and probable impossibility of carrying out the Patton scheme, and recommended against it, giving as his opinion that it was too dangerous to warrant its adoption. He recommended that the cribs be cleaned out and filled with concrete. Although recommending against the repair work, he submitted the lowest bid for it, and received the award.

The work of sinking the annular caissons and coffer-dams went forward in the fall of 1898, and was completed early in the summer of 1899.

At Pier 4, a timber caisson, about 50 by 70 ft., was constructed around the old pier, the intention being to carry out the work, as nearly as practicable, in accordance with Mr. Tullock's proposal of August 17th, 1898 (Case 2, Appendix B).

Two rows of piling were driven around the old pier, about 8 or 10 ft. apart, and were capped and cross-capped; the caisson and crib were built on top of these caps to a height of 12 ft. Large screws, 16 ft. long, were then used to lower the crib until it floated. The crib projected 4 ft. above the water, and the cutting edge floated about 4 ft. above the bed of the river; sand was used to fill the crib to sink it, while men were raising timber on top of the crib.

On March 1st, 1899, with the cutting edge on the bed of the river at an elevation of 11 ft. below zero, air was forced into the working chamber and excavation was made through the following material:

From Elevation 11 to 17..........Sand.
From Elevation 17 to 23..........Rip-rap and sand.
From Elevation 23 to 41..........Sand.

On April 2d, 1899, at 41 ft. below low water, hard shale was struck near the center of the up-stream end. From 41 to 45 ft. below zero, excavation was made through very hard black shale on the north side and at the up- and down-stream ends. This rock was nearly as hard as granite, requiring dynamite to make the excavation.

At an elevation of 45 ft., the inside cutting edge on the south side was landed on rock, except about 10 ft., where the rock was 6 in. below the edge. The rock under the outside cutting edge on the south side was from 12 to 13 in. below it. However, the sand was taken out down to the rock, and the working chamber was sealed with from 2 to 4 ft. of concrete, depending on the depth of rock.

The crib was built 58 ft. high, the top being 13 ft. above low water when the caisson was sealed. Later, the top of the crib was disconnected, from 6 to 7 ft. below low water, after concrete had been placed between the old and new cribs. The sealing of the working chamber was completed on April 24th, 1899.

The river began rising very rapidly on April 22d, and on the 23d a 7-ft. rise was reported at Fort Smith. This reached Little Rock a few hours after the work of sealing the working chamber had been completed, the water rising over the top of the crib. Work was suspended from April 25th until June 30th, and then two pumps and an air-lift were used to remove the water between the old and new cribs, and men commenced excavating the loose sand and sediment from around the old pier, but no effort was made to go below the depth uncovered by the pumps, or to remove the silt. The silt and sand were cleaned out to a depth of 5 ft. on the up-stream and 7 ft. on the down-stream end below the top of the old crib. Investigation of the timber in the old crib disclosed crushing on the north side, but only to the extent of compressing the timber, and not dividing the grain. The timber was good and sound.

A great deal of the 3-in. sheeting was found to be torn off, and it was seen that the spaces between the timbers of the crib were filled with sand and loose rock. The sheeting which had been torn off was replaced. It was also observed that the deck or top of the old crib dipped from 1 in. to 1½ in. from a point near the center on the south side of the pier toward the up-stream end.

On July 1st, while the loose material between the new coffer-dam and the old crib was being excavated, it was discovered that the pier had settled about ½ in. This led to the immediate abandonment of the excavation, but the spaces on the north and west sides were driven full of piles, with a penetration of about 15 ft., and the space already excavated was filled with concrete up to the top of the old crib. None of the material within the old crib was removed, and no effort was made to grout it or to fill in the vacant spaces with concrete. In fact, the sheeting that was replaced effectually prevented the concrete from flowing into the vacant spaces within the crib. The concreting around the old pier was completed on July 6th, 1899, and both walls of the annular coffer-dam were then removed, down to 6 or 7 ft. below low water.

The location of the new annular caisson and coffer-dam around Pier 4, with reference to the old crib, together with the general construction of both, is shown by Fig. 5, which also shows in cross-section the concrete, from 3 to 5 ft. thick, that was placed between the old crib and the new coffer-dam. The annular caisson was about 50 ft. wide and 70 ft. long, the sides were 10 ft. apart, and spaces from 6 ft. to 7 ft. 6 in. wide were left between the old and new work.

The work at the pivot pier was similar in character, except that the new caisson was 70 ft. square and the concrete about 15 ft. thick, the extra depth having been caused by scour and not by excavation. (Fig. 5 does not show correctly the pier top at this time (1898), but as it appeared several years later, after additional movement had taken place.)

Mr. Tullock considered the removal of the material between the new and old caissons to be very hazardous, and he continued to impress this opinion on the officials of the Railway Company. He was so much impressed by it that he continually recommended and urged that only part of the material be removed and that the remainder be confined by placing concrete over it. Unfortunately, the Railway Com-

pany was guided by his advice in this matter. Two of Mr. Tullock's letters to Mr. Fisher, relative to this matter, dated November 24th, 1898, and May 16th, 1899, are printed in full in Appendix C.

It is to be regretted that, after having incurred the great expense of sinking the annular caissons, more benefit was not derived from them. The scheme of jacking up the piers and underpinning them

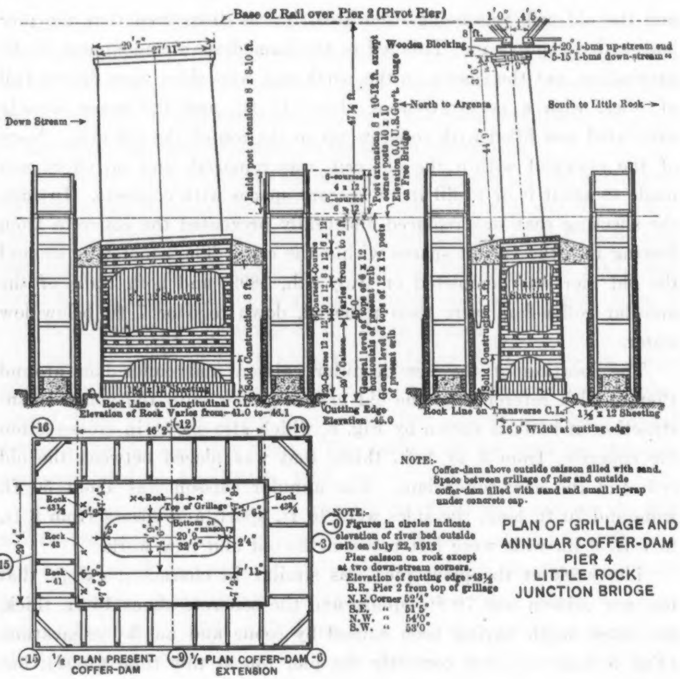


FIG. 5.

appears to have been abandoned early, as no reference to these matters can be found in the files during the work. Neither was any effort made, by exploring under the cutting edges of the new caissons, to determine the depth to which the original caissons had been sunk.

During May, 1899, the concrete having been placed around the pivot pier, the wheel-tread under the drum of the draw-span was leveled up

and underpinned with concrete, the minimum rise being 2⅜ in. and the maximum 6½ in.

At the same time, but prior to the placing of the concrete around Pier 4, the spans on that pier were also leveled up and carried on oak blocking in place of the bed-plates, the spans being raised from 1¼ to 6 in. This work was done too soon, as a slight settlement of Pier 4 during the work threw the track out of line.

The pier tops had moved around so much, and their upper surfaces were so small, that the trusses could not be placed on a straight line. The trusses had been built with extra wide clearance to provide for highway traffic, and this permitted considerable latitude in the location of the track. The trusses were permitted to remain out of line, and the track was placed on a tangent, being nearer to one truss than to the other where the spans were out of line. The location of the center line of the track with reference to the track stringers and trusses is shown on Plate I.

Further Movement of Pier 4.

On account of the large quantity of concrete placed around the pivot pier, and the depth to which it reached, 15 ft. below the top of the crib, no further movement has been detected in this pier. However, the movement of Pier 4 was not arrested, but as the settlement and movement continued, a policy of "watchful waiting" was followed for several years. The situation again became critical early in 1906, when Pier 4 had moved so far that one of the spans was in imminent danger of falling off.

At that time J. C. Bland, M. Am. Soc. C. E., was retained as Consulting Bridge Engineer of the Missouri Pacific Railway, and was asked to look into the matter and pass on it. He arranged to have John N. Ostrom, M. Am. Soc. C. E., visit the bridge and report on it. That portion of Mr. Ostrom's report having reference to the foundations is as follows:

"The roller nests at north end of span, which is next to the draw-span on the north, are badly shifted to the south, so that two of the rollers do not bear.

"The main trouble with the bridge is in the foundations, and, like the Baring Cross Bridge, about one mile above, there are no original records showing the exact conditions of the bearings on the bottom.

The pivot pier, Pier 3 and Pier 4, on the north side of the pivot, are all tilted up stream and also sideways.

"The pivot pier and Pier 4 have been protected by concrete additions extending clear around the base on the outside and reaching down into the rock. It is believed that the tilting of the pivot pier, at least, has been arrested.

"The piers also have well-defined vertical cracks extending through several courses, indicating uneven bearing on the bottom, but the cracks are not bad enough to endanger the structure at present.

"The south abutment has a deep crack in the up-stream wing-wall, and the north face next to the river has several bands, one above the other, held by bolts which pass south into the abutment.

"Pier 7 on the north shore is cracked, and banded with tie-rods running through yokes on the up- and down-stream ends.

"The most important matter at present is to make careful measurements at least once a month on all the piers and abutments with a view of determining whether or not the tilting and cracking has stopped. If these measurements show that the masonry has become fixed in position, it may be assumed safe for the life of the present superstructure, but it would not be suitable for a new bridge designed under present specifications.

"If the cracks in piers increase, they should be banded, and the cracks should be pointed as soon as practicable, to exclude water.

"Owing to the tilting of foundations, the surface and alignment of the track is very bad on the bridge. As the piers have tilted up stream the track has been shifted down stream to keep the alignment as good as possible until the down-stream stringers are carrying considerable more than half the load. Furthermore, the clearance between the train and the down-stream truss has been reduced by the amount of shifting of the track."

"Jno. N. Ostrom."

"To Mr. J. C. Bland.

"May 21, 1906."

In transmitting Mr. Ostrom's report to M. L. Byers, M. Am. Soc. C. E., then Chief Engineer of Maintenance of Way, Mr. Bland wrote in part as follows:

"June 4, 1906.

"M. L. Byers.

"One can hardly tell much about the masonry tilting. The plans you sent show the tilting but do not say the character of foundation, nor whether the condition is becoming worse. You will notice that Mr. Ostrom says:

"'It is believed that the tilting of the pivot pier, at least, has been arrested.'

"Who is it that says so, and on what grounds?

"I think you should use slow speed over the bridge, and also I would prohibit the stopping and starting of trains on bridge, if such an order is practicable.

"J. C. BLAND."

Following this report no immediate action was taken.

One of the first duties assigned to the writer when he entered the Bridge Department of the Railway Company, late in 1907, was an inspection of this bridge in order to determine how to maintain it in safe condition. At that time, the location of the pedestals of the spans with reference to the coping courses on Piers 3, 4, 5, and 6, was as shown in Fig. 6. Pier 4 had moved out so far from under the shoes that the center of the end pins was almost exactly over the edge of the timber blocking under the shoe; the edge of the shoe overhung the edge of the pier, as shown in Fig. 7.

PLAN SHOWING RELATIVE POSITIONS OF TRUSS PEDESTALS AND TOP OF PIERS LITTLE ROCK JUNCTION BRIDGE DEC. 1907

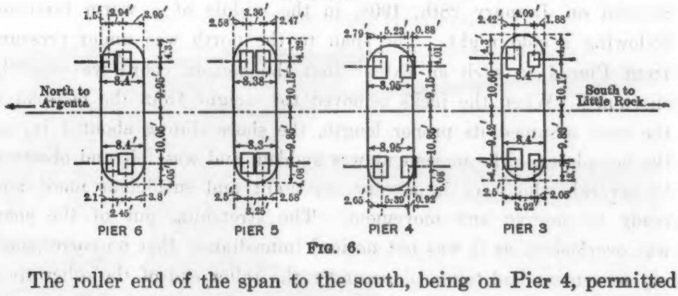


FIG. 6.

The roller end of the span to the south, being on Pier 4, permitted the pier to move out from under. The fixed end of the span to the north being on Pier 4, the thrust caused by the leaning of the pier in that direction had crowded the roller end of this span on Pier 5 hard against the fixed end of the next span. This thrust, added to the similar, though less, defective construction of Pier 5, caused the latter to lean north, crowding the roller end of the next span to the north, hard against the fixed end of the north span on Pier 6, bringing three spans in hard contact. On hot days, when the bridge was not loaded, the chord bars in the end panels of these spans could be seen to be more or less buckled, but they always straightened out

under trains, undoubtedly because the elasticity of the timber cribs permitted the piers to move back and forth.

Immediately following his first visit to the bridge, the writer worked up plans for placing a nest of I-beams under the truss shoes at each end of Pier 4, so that the overhanging ends of these beams would afford support for the end of the next span south to keep that span in the air instead of in the water. Fig. 9 shows the arrangement of I-beams for this purpose. There was only sufficient depth for 15-in. I-beams at the high end of the pier, but at the low end, there was sufficient depth for 20-in. beams, the difference in height being due to the settlement.

The plans first contemplated driving piles on both sides of Pier 4 to be used for jacking during the placing of the I-beams, but, on account of the extreme length of the piles required (from 80 to 90 ft.) and the hazard attending their maintenance in the Arkansas River, it was decided to avoid their use and jack from the pier top.

The jacking arrangement was set in place and jacking was commenced on January 28th, 1908, in the middle of a warm forenoon following a cold night. The span to the north was under pressure from Pier 4 to such an extent that the bottom chord was slightly shortened. When the jacks removed the weight from the bed-plates, the span assumed its proper length, the shoes sliding about 1 in. on the bed-plates. The movement was sudden, and was felt and observed by several, who were in a very expectant and suspicious mood and ready to observe any movement. The stretching out of the span was overlooked, as it was not noticed immediately that no corresponding movement had taken place under the roller end of the other span on the same pier. The pier had such a bad record that all the men engaged on the work believed it had suddenly moved and was going to fall, and the forces, led by the foreman, fled from the structure, headquarters being advised by telegram as follows:

"Began jacking bridge this A. M.; raised 1 in. when Pier 4 moved north 1 in. Will be necessary to yoke Piers 3 and 4 to span before it will be safe to handle driver for falsework. It will be necessary to put spans on falsework before we can resume traffic."

As an emergency matter, to retard the movement of Pier 4 and to avoid any such scare during the work, the yokes and rods shown

FIG. 7.—TRUSS SHOE OVERHANGING BEARING.

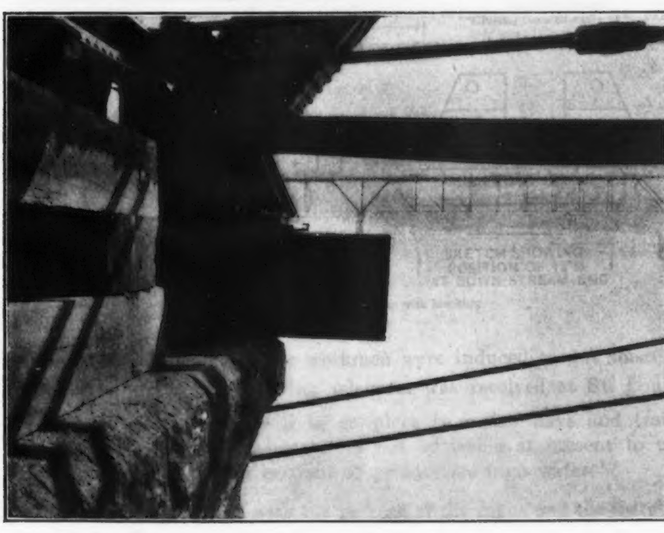
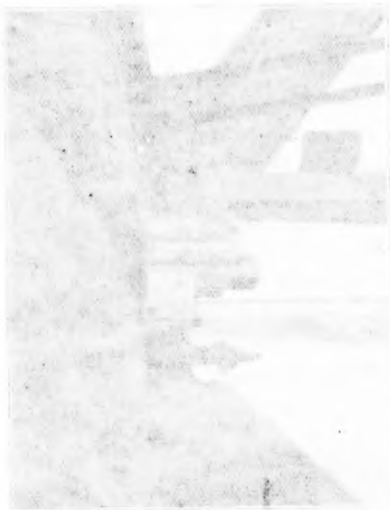
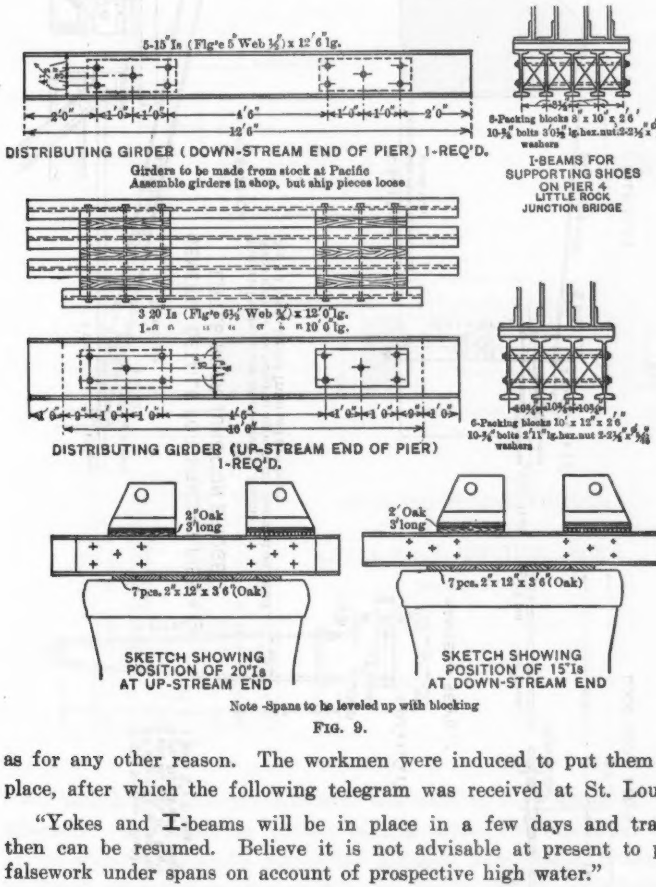


FIG. 8.—TRUSS SHOE ON I-BEAM GRILLAGE.



by Fig. 10 had been designed, but the conclusion was reached that they would not be necessary. After the apparent movement of the pier, however, it was impossible at first to induce the workmen to return to the structure, and arrangements were completed to make the rods, etc., immediately, and to apply them—for their moral effect as much



Note -Spans to be leveled up with blocking

FIG. 9.

as for any other reason. The workmen were induced to put them in place, after which the following telegram was received at St. Louis:

"Yokes and I-beams will be in place in a few days and traffic then can be resumed. Believe it is not advisable at present to put falsework under spans on account of prospective high water."

They then went ahead with the jacking of the spans and the installation of the I-beams. These were finally placed and the bridge was

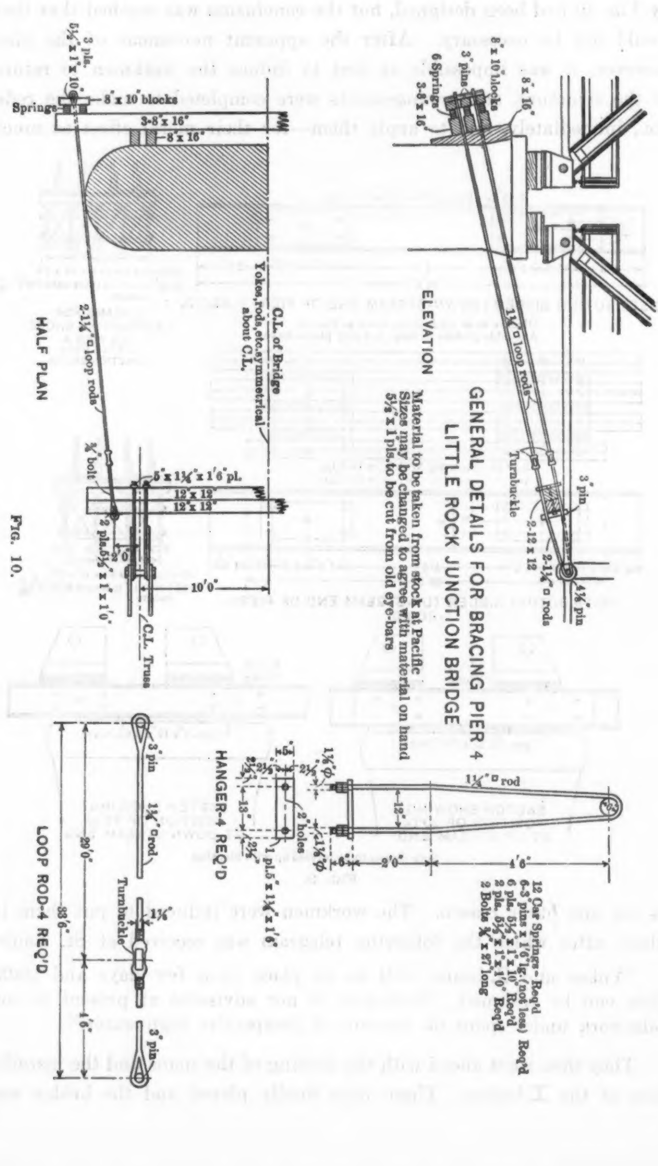

GENERAL DETAILS FOR BRACING PIER 4
LITTLE ROCK JUNCTION BRIDGE
Material to be taken from stock at Pacific
Sizes may be changed to agree with material on hand
5/8 x 1 pls. to be cut from old eye-bars
ELEVATION
HALF PLAN
HANGER-4 REQ'D
LOOP ROD-4 REQ'D
Turnbuckle

FIG. 10.

restored to service on February 1st, 1908, 4 days after the commencement of the jacking.

After placing the I-beams, the spans were shifted slightly to remove their interference, and the rollers were cleaned and oiled. The yokes were left in place, and instructions were issued to have them tightened

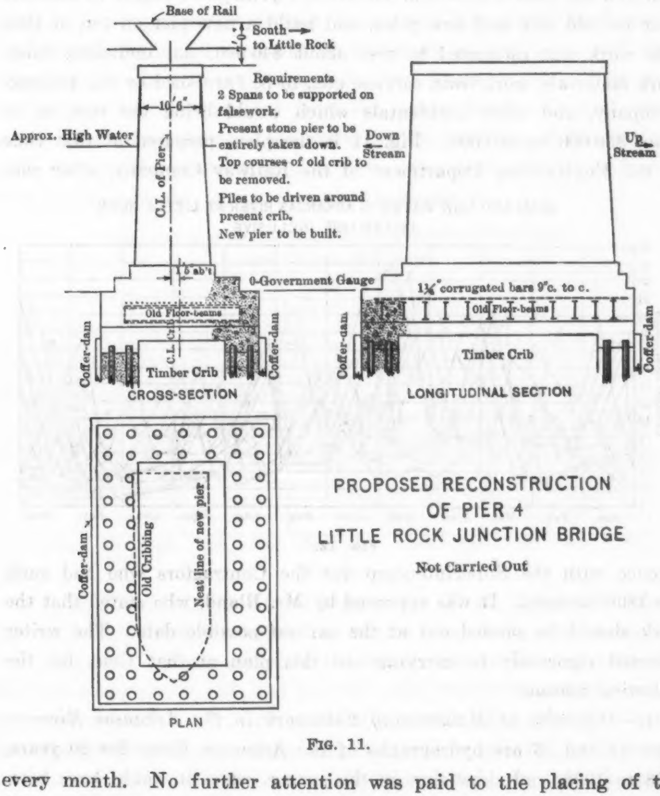


FIG. 11.

every month. No further attention was paid to the placing of the falsework. The condition of the bearing under the expansion end of the span to the south, before the I-beams were placed, is shown by Fig. 7. Fig. 8 shows the condition afterward. The bearing having been made safe, the matter of permanent repairs was then given further study.

Plans for Reconstruction of Pier 4.

Late in 1907 tentative plans had been made to replace Pier 4 by a larger one. It was planned to place the two spans resting on Pier 4 on falsework, take down the masonry pier to the top of the crib, drive as many piles as possible in the annular space between the old crib and the 1898 coffer-dam, construct a grillage of beams in concrete over the old crib and new piles, and build a new pier on top of this. The work was estimated to cost about $40 000, not including falsework materials, work-train service, etc., to be furnished by the Railway Company, and other incidentals which would bring the cost up to from $50 000 to $60 000. Fig. 11 is the plan, prepared at that time by the Engineering Department of the Railway Company, after con-

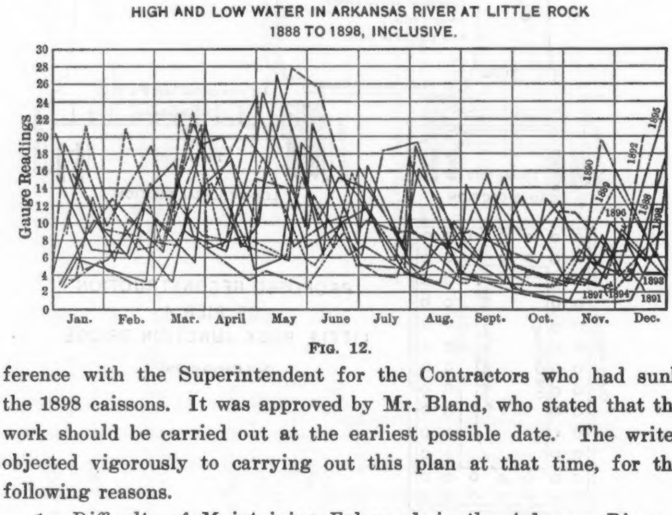


Fig. 12.

ference with the Superintendent for the Contractors who had sunk the 1898 caissons. It was approved by Mr. Bland, who stated that the work should be carried out at the earliest possible date. The writer objected vigorously to carrying out this plan at that time, for the following reasons.

1.—Difficulty of Maintaining Falsework in the Arkansas River.—Figs. 12 and 13 are hydrographs of the Arkansas River for 20 years, 1888 to 1908, and show that in the spring, when it would have been necessary to maintain two 253 ft. 4-in. spans on falsework with 80-ft. piles in the middle of the river, the floods are greatest. They also show that sudden rises can be expected in any month of the year, and are greatest in spring and least in early fall. Rises in the Arkansas are invariably rapid, and are characterized by excessive

quantities of drift, some of it whole trees, 80 or 90 ft. long, from 2 to 3 ft. in diameter and full of branches, and also by deep and rapid scour, down to rock, of the fine sand and silt forming the bed, especially around obstructions of any nature. In the past the Arkansas has taken heavy toll in falsework and spans, and there is hardly a bridge across the river where there has not been some trouble from this source. It is practically certain that great loss would have been suffered had the spans been put on falsework at that time, and the danger would have been increased by the delays in rebuilding the pier, which could be expected from the spring floods. In fact, this consideration has influenced the writer, in the handling of this bridge up to date, to the extent that piles have never been driven for falsework purposes in the repairs that have been made.

HIGH AND LOW WATER IN ARKANSAS RIVER AT LITTLE ROCK
1898 TO 1908, INCLUSIVE.

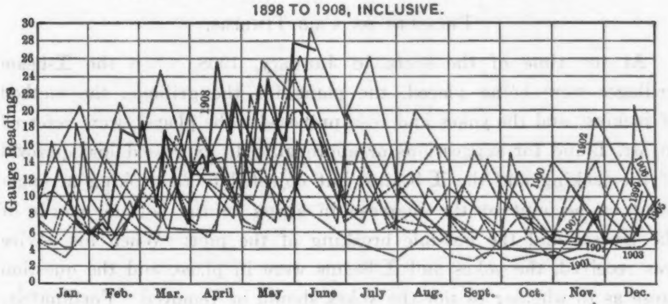


FIG. 13.

2.—No Provision for Strengthening the Weak Crib.—The plan did not provide for any correction of the weak crib, and it would have been impossible to calculate the distribution of load, as between the very compressible crib and the rigid pile foundation, which would have surrounded it. It was practically certain that from the very first the proposed piles surrounding the crib would be badly overloaded, and failure of some sort could be expected. In fact, the proposed pier was so much larger than the old pier that, if the load could have been distributed uniformly over the base, the weak crib would have been required to carry more load than before.

3.—Better results could have been obtained at greatly reduced expense by driving piles as proposed and building up on them a

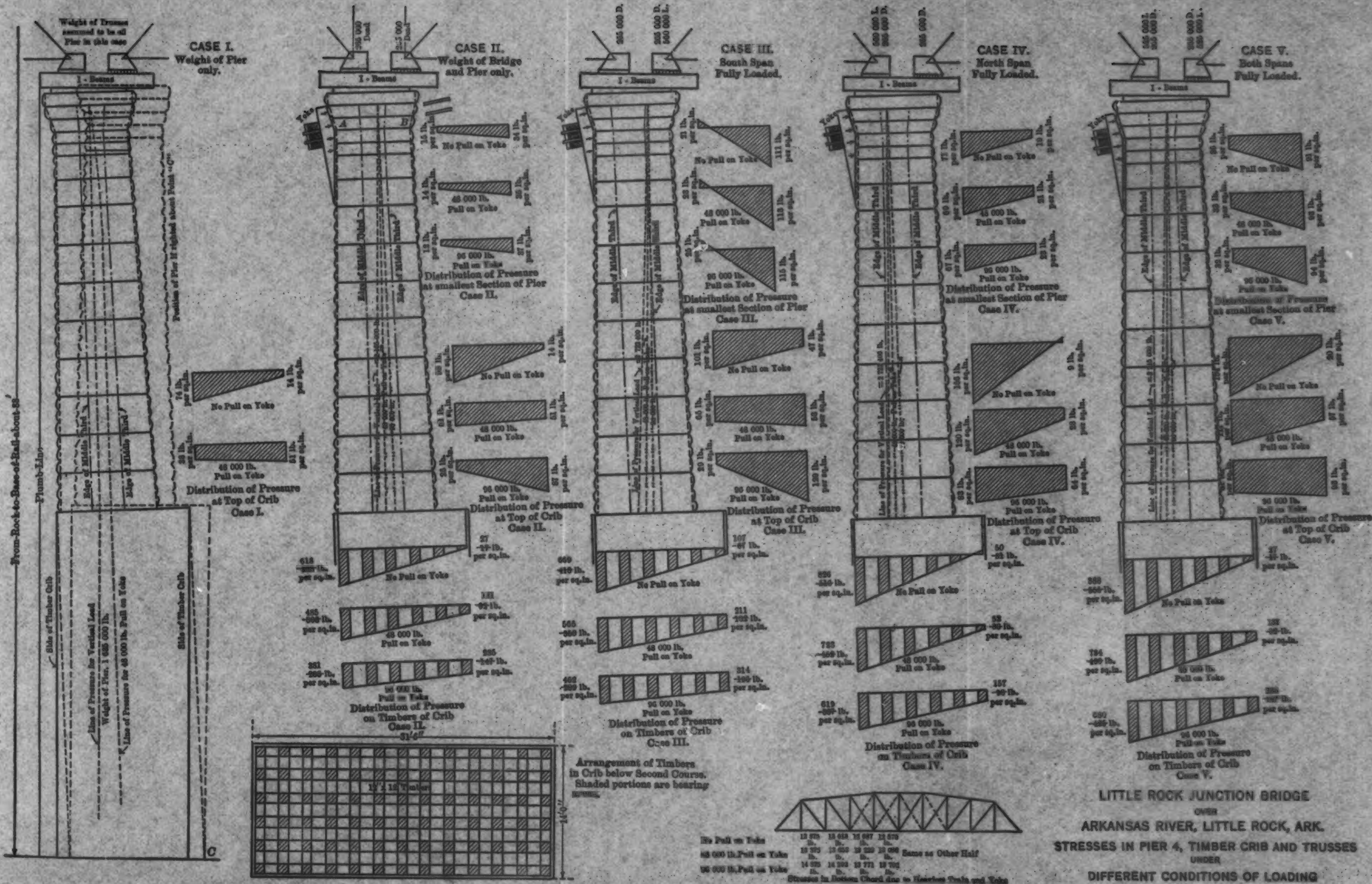
reinforced concrete shell enclosing the old pier, thereby saving the latter and avoiding the cost of its renewal, all the cost and hazard of falsework, and, to a great extent, the uncertainty as to the distribution of the loading. The art is sufficiently advanced to insure absolutely efficient bond between the new encasement and the old masonry (the old was rough rock-face). The reduced uncertainty as to the distribution of the load is due to the fact that from the first the crib would carry its maximum load and be fully compressed during the construction of the encasement, so that the new support would work well with the old.

These three reasons were sufficiently potent to prevent the adoption of this scheme, but it was not possible for the writer to secure in the near future authority to go ahead with the driving of additional piles and the encasement of the pier.

Pressure on Crib Timbers.

At the time of the scare in January, 1908, when the I-beam grillages were being placed, the plans for the grillage, the method of jacking, and the yokes and rods proposed to be placed, were referred to Mr. Bland for suggestions or approval. He expressed his approval of the jacking and the I-beams, but objected to the yoking arrangement on account of the increase in stress in the bottom chords of the trusses and the possible breaking of the pier. When his advice was received, the yokes and I-beams were in place, and the question arose as to whether or not the yokes should be removed. Fortunately, no one was prepared immediately to assume the responsibility of removing them, and as the writer had suggested putting them on, he was requested to show cause why they should be permitted to remain. Rough figures had previously been prepared, showing the pressure on the crib timbers and the effect of the yokes, but to show better the beneficial effect of even a moderate pull near the top of the pier, the lines and intensities of pressure were determined for all probable conditions of loading and assumed stresses in the yokes. Plate II shows the results, and indicates that even a moderate pull at the top of the pier would be of considerable advantage. The nuts were tightened from time to time, and the springs were kept at such a pressure as to maintain a pull of about 50 000 lb. As the end panels of the bottom chords of the trusses had 48 sq. in.

PLATE II.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXIX, No. 1385.
SMITH ON
REPAIRING BRIDGE PIERS.



LITTLE ROCK JUNCTION BRIDGE
OVER
ARKANSAS RIVER, LITTLE ROCK, ARK.
STRESSES IN PIER 4, TIMBER CRIB AND TRUSSES
UNDER
DIFFERENT CONDITIONS OF LOADING

of cross-section, figures of 48 000 and 96 000 lb. were assumed in the calculations, for the amounts of pull corresponding to 1 000 and 2 000 lb. per sq. in., respectively, the latter figure having also been used in the calculations to ascertain the results if the pull should reach double the amount assumed. The results showed conclusively that the trusses and the masonry pier were not adversely affected by the pull of the rods. The maximum stress in the bottom chord under the heaviest loading was about 13 000 lb. per sq. in. before the application of the rods and about 15 000 lb. per sq. in. afterward, an amount well within safe limits.

In order to avoid damage to the pier by the concentration of the pull of the yokes, bearing timbers had been cut to fit the shape of the rock face of the pier and placed in contact with the pier for a height of about 10 ft., resulting in spreading the greater part of the pull over four or five courses.

Fig. 10 shows the yokes and rods. The arrangement consisted of three 8 by 16-in. timbers laid horizontally and blocked against the side of the pier toward which it was leaning, the ends projecting beyond the pier ends and being attached at each end to two 1¼-in. round rods with upset ends. The rods passed close by the pier and extended up to the first panel points of the trusses, where, by the timbers and U-rods, the pull was transmitted to the bottom chord of the trusses. Heavy car springs were introduced at points of bearing, partly to take up the expansion and contraction. The other end of the span to which the yoke had been connected was attached to Pier 3 by a duplicate of this arrangement.

The line of pressure is well within the middle-third for all cases except when the south span only is fully loaded, under which condition the line is outside the middle-third for the top 20 ft. The compression on one side and the tension on the other side did not reach intensities that would cause any alarm, however, and the effect of the pull on the rods was only barely appreciable at that section of the pier.

Alarm had been felt, and the opinion was freely ventured, that the pier was held standing by the weight of the trusses. To dispel this alarm, Case I was chosen; as might have been expected, it shows the fallacy of that opinion and the entire safety of the pier when standing alone.

The records indicated that the crib timbers were 2 ft. 6 in. apart, the old plans showing six longitudinal and thirteen transverse timbers in alternate courses, the bearing areas being the intersections shown cross-hatched in Plate II, a total of 78. The timbers were assumed to be 12 in. wide, thus giving a bearing area of 78 sq. ft. The diagrams showing the distribution of pressure on the timbers of the crib were prepared on this basis, on the assumption that the timbers carried the entire load, and the intensities that have been crossed out were the result. Several years later, when the crib was uncovered, it was learned that there were only five longitudinal and eleven transverse timbers in the alternate courses, and that the center intersection had been omitted in order to make room for the shaft, giving 54 intersections instead of 78. The timbers were found to be of various sizes, from 11 to 12 in. wide, so that the total bearing area was in reality about 50 sq. ft. It was also learned that the timbers carried practically all the load. Calculation with reference to the true conditions would have increased their intensity about 60%, or to the figures given above those crossed out. The maximum pressure of 555 lb. per sq. in. on one side, with 17 lb. per sq. in. on the other, calculated from the information available when the study was first made, easily accounted to the writer for the uneven settlement of the pier, but he had few adherents. Had the correct figure of about 888 lb. per sq. in. in cross-bearing been available at that time, all might have been convinced.

The position of the cross-section of the pier above the crib in Plate II was plotted from very careful measurements in 1908 from the plumb-line shown at the left. The position of the crib and caisson, shown in full lines, was estimated on the assumption that the tilt of the pier was caused by the crushing of the timbers uniformly distributed throughout the height.

The position which the crib and caisson would have occupied, on Mr. Patton's theory of settlement into the foundation and no uneven crushing of timbers, was found by projecting the sides of the crib at right angles to the top, the location and slope of which had been measured. The right-hand cutting edge of the caisson would have been at *C*, and if, by jacking under the other edge, the pier had been revolved about *C* to plumb position, it would have reached the position shown by the dotted lines. The absurdity of that theory is indi-

cated by the position the top of the pier would have occupied with reference to the pedestals of the trusses. The span to the right was never any farther to the right, its position being controlled by its proximity to the draw-span and the proximity of the latter to the south abutment. There would not have been room for the south pedestal of the next span to the north, shown at the left on Plate II.

The figures shown on Plate II convinced all concerned that there would be no danger in permitting the yokes to remain in place, and orders were issued to keep them tight. No fear was felt as to the effect of the pull on Pier 3, at the other end of the span, to which the yokes had been applied, because the south side of that pier had been built plumb, the entire batter being on the north side, the natural lean of the center of the pier being opposed to the direction of pull; in fact, the pull really improved the distribution of stress in Pier 3 in a manner somewhat similar to that at Pier 4, but less pronounced.

Fig. 3 shows Pier 4 after the yokes and I-beams had been applied, and on Fig. 14 there are transverse and longitudinal sections through the pier showing the relations between the yokes, the I-beams, and the pier.

Steel Bent Constructed at Pier 4.

In 1908, some one got the idea that it would be of advantage to provide further rigid support for the overhanging ends of the I-beams on the high side of the pier by building a concrete footing on the wide edge of the crib up to low water and placing a steel bent on top of that footing, as shown by Fig. 15. The I-beams under the pedestals were to be shifted far enough south to get bearing on the bent, and the spans were to be shifted as far south as possible in order to bring the load over toward the high side of the crib. This method might have been very effective had it been possible to move the spans any great distance, but, on account of the proximity of the south span to the draw, and of the north end of the north span to the edge of Pier 5, only a 6 to 10-in. movement could be made, and the effect on the line of pressure was barely perceptible. However, the work was done in the fall of 1908, by the Missouri Valley Bridge and Iron Company, at a cost of about $2 000. Fig. 16 gives a general idea of the manner in which the work was carried out.

Holes were drilled in the top of the crib and in the side of the pier at low water, and bent reinforcing rods were placed in them. A form

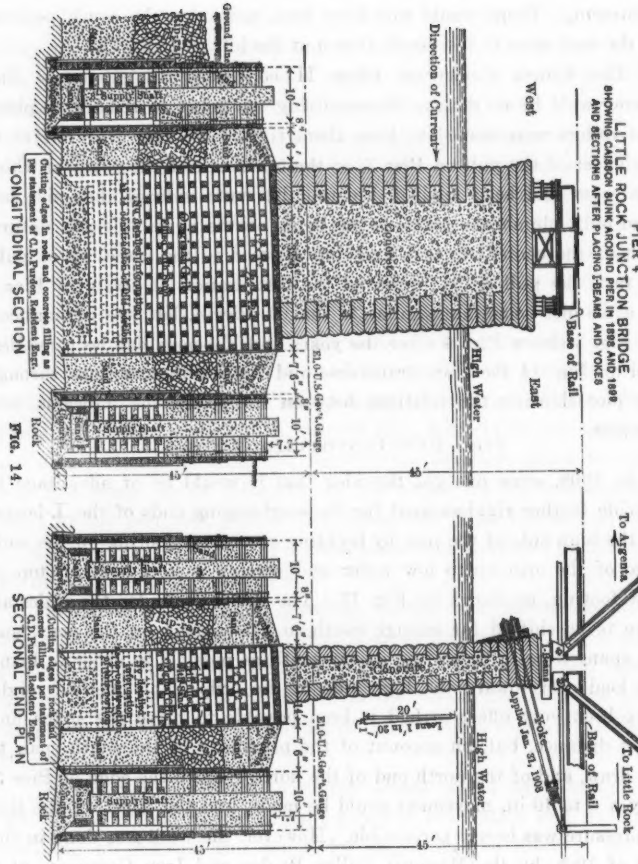
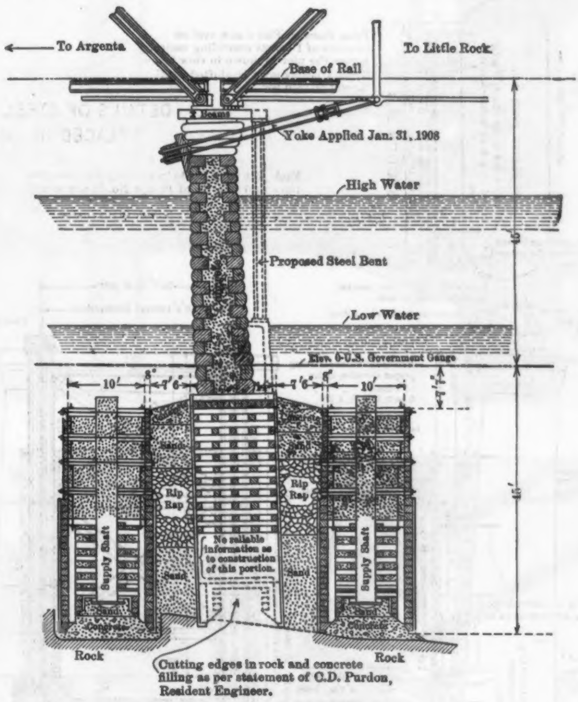

PIER 4
LITTLE ROCK JUNCTION BRIDGE
SHOWING CAISSON SUNK AROUND PIER IN 1898 AND 1899
AND SECTIONS AFTER PLACING I-BEAMS AND YOKES
West
East
Direction of Current
Base of Rail
High Water
Concrete
Ground Line
Sand
Rip-rap
3' Supply Shaft
El.0-U.S.Gov't.Gauge
Rock
45'
Cutting edges in rock and concrete filling as per statement of G.D.Purdon, Resident Engr.
LONGITUDINAL SECTION
To Argenta
To Little Rock
I-Beams
Yoke
Applied Jan. 31, 1908
20'
SECTIONAL END PLAN

FIG. 14.

was lowered through the water to the top of the crib, and the concrete was deposited by a bottom-dump bucket. A steel bent, made of old top-chord sections, was then placed on this footing and held to the pier by U-shaped reinforcing rods set in grouted holes. The col-



SECTIONAL END PLAN
PIER 4
LITTLE ROCK JUNCTION BRIDGE
SHOWING CAISSON SUNK AROUND PIER IN 1898 AND 1899.

FIG. 15.

umns were then encased in concrete up to high water, and a reinforced concrete shell was constructed between them to ward off drift.

Then the I-beams and spans were shifted as far as possible, in order to throw the greatest possible load on the new bent. The work was carried out carefully and expeditiously, and offered no unusual

difficulties. Fig. 17 is an end view of Pier 4, showing the new bent in place.

Following this work, no appreciable movement in the direction of the bridge was apparent for several months, but the movement at

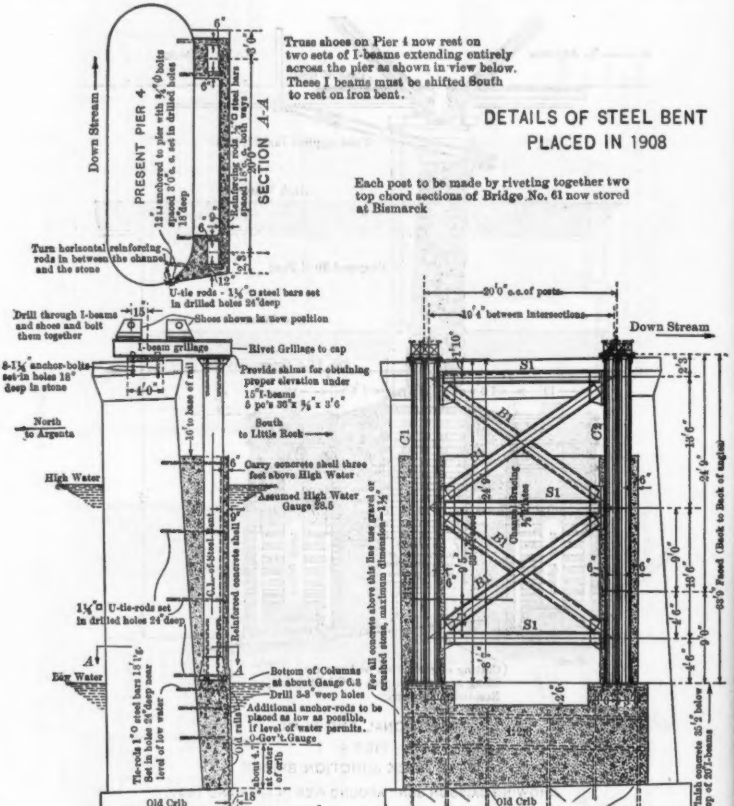


FIG. 16.

right angles to the bridge appeared to increase its rate, indicating that the work had no appreciable effect.

The movement of the pier was greatly dependent on the scour line. When the current shifted away from the pier, the bed of the

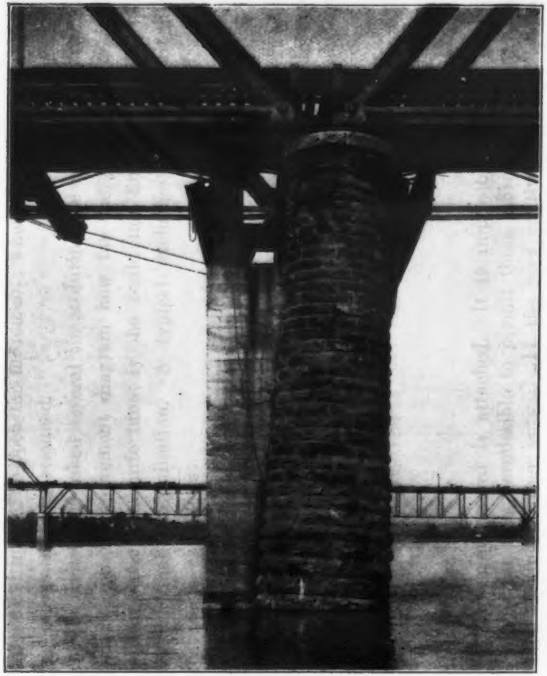


FIG. 17.—PIER 4, SHOWING STEEL BENT ENCASED IN CONCRETE.

FIG. 18.—PIER 3 AFTER WORK WAS COMPLETED.

river filled up and stopped the movement; when the opposite was true, the movement was resumed.

The writer was asked several times during the next year or two to indicate on the movement diagram how far the pier could go and remain safe, but, unfortunately, he could not find any method of making that determination. A typical exchange of notes on that feature is as follows:

"ST. LOUIS, August 7, 1909.

"MR. HALE:

"I presume you are in touch with the movement records which Mr. Ford is keeping, and would like to have your opinion as to how far it is permissible to permit these movements to continue before taking further action.

"M. L. BYERS."

"8-17-09.

"MR. HALE:

"Movement record is attached. It is impossible for any one to say how far it is permissible to permit these movements to continue before taking further action. If the pier continues stationary, no action whatever is necessary, as the piers are all perfectly safe. If the movement resumes we must take action.

"C. E. SMITH."

OTHER PLANS FOR OVERCOMING TROUBLE AT PIER 4.

Following the work in the fall of 1908, the observations were continued, and the matter of proposed permanent repairs or reconstruction was kept constantly alive for 3 years, during which time scheme after scheme was evolved and considered, but nothing was started until the fall of 1911. As a number of the schemes were ingenious and interesting, some of them will be described.

It was proposed to take down Pier 4 to the top of the crib, sink to rock along each side of the crib 12 or 15-in. steel cylinders filled with concrete, by the usual well-drilling methods, and build a structural steel tower on top of the cylinders and crib, as shown by Fig. 19.

This was first studied on the basis of having the load divided equally between the cylinders and the crib, which would have reduced the maximum pressure on the timbers from 555 to 150 lb. per sq. in., on which basis the estimated cost, including the falsework for supporting the spans and taking down the old pier, was $32 200.

It was realized, however, that on account of the great stiffness of the cylinders and the steel tower, and the great elasticity of the crib, it would be impossible to control the distribution of load among the supports, and, in any event, most of the load would go to the cylinders. If the cylinders were made large and numerous enough and the steel tower strong enough to carry the entire load, the cost was estimated at $43 200.

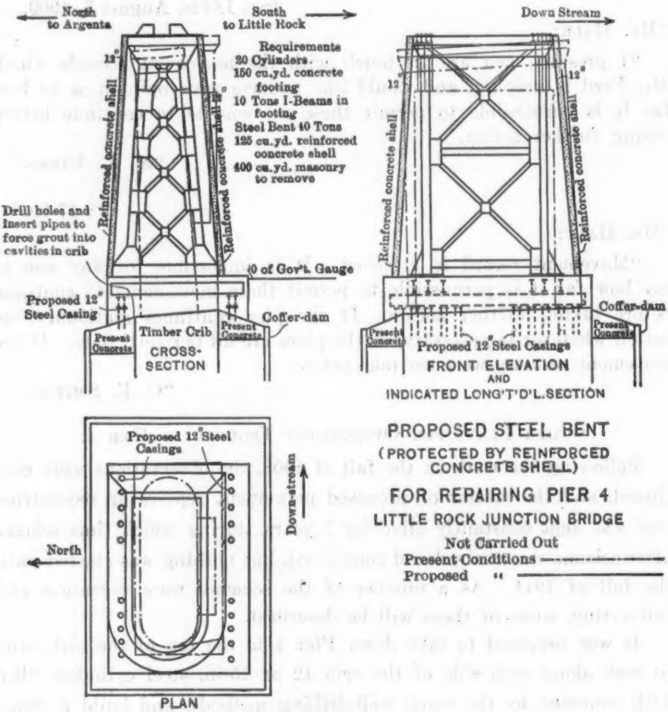


FIG. 19.

On account of the high cost and the uncertainties attendant on this scheme, it was abandoned.

The success obtained in placing the steel bent on the south side of the pier, and the small expense incurred, encouraged the proposition of building another steel bent on the north side. As there was no

projection of the timber crib on which to rest such a bent, and as the north edge was badly overloaded and thought to be crushing, it was proposed to sink a row of steel cylinders on the north side on which to rest the bent. This scheme also necessitated the use of longer and heavier beams on top of the pier, in order to distribute the load out to the bents. On account of the pedestals of the trusses being so far on the pier, it was almost impossible to study out any plan that would reduce the load on the pier an assumed amount, but, by changing the point of contact between the I-beams and the top of the pier, it was found possible to throw about 250 000 lb. on the new bent, thereby reducing the maximum pressure on the timbers from 555 to 477 lb. per sq. in. This was estimated to cost about $10 000. The scheme was abandoned on account of its uncertainty.

Of course, it would have been possible to have taken the weight entirely off the old pier and provide cross-girders sufficiently strong to throw the entire load on the steel bent, but the extra expense and uncertainties of this scheme ruled it out.

In the meantime the writer had worked up the plan shown on Fig. 20, which contemplated driving piles to rock by jetting, within two sides of the annular space between the crib and the coffer-dam, toward which the pier was moving, the plan being to remove the concrete cap a section at a time, drive several piles, and replace the concrete as a sub-footing course, and incidentally to preserve the side support afforded by the concrete. To accomplish this it was proposed to build a puddle coffer-dam around the pier within the 1898 coffer-dam in order to hold the water out against about a 10-ft. head, and to force as much concrete and grout as possible into the crib.

After grouting the crib, driving the piles, and restoring the concrete cap, it was proposed to construct, around the old pier and on top of the projections of the crib and on the new piles, a solid reinforced concrete structure which would materially enlarge the base, restore the symmetry, and effectually protect the masonry for all future time.

The rough rock-face and round ends of the pier made it particularly suited to encasement, but, in order to make the bond as intimate as possible, it was planned to drill through the body of the pier at several places near the bottom and place second-hand eye-bars through the pier, with suitable anchors at each end to be embedded in the con-

crete shell; also to curve the horizontal reinforcing rods continuously around the ends of the pier, and to replace the top by a new thick reinforced concrete cap. This plan had the advantage that no false-work would have been necessary, and it could have been carried out for $12 000.

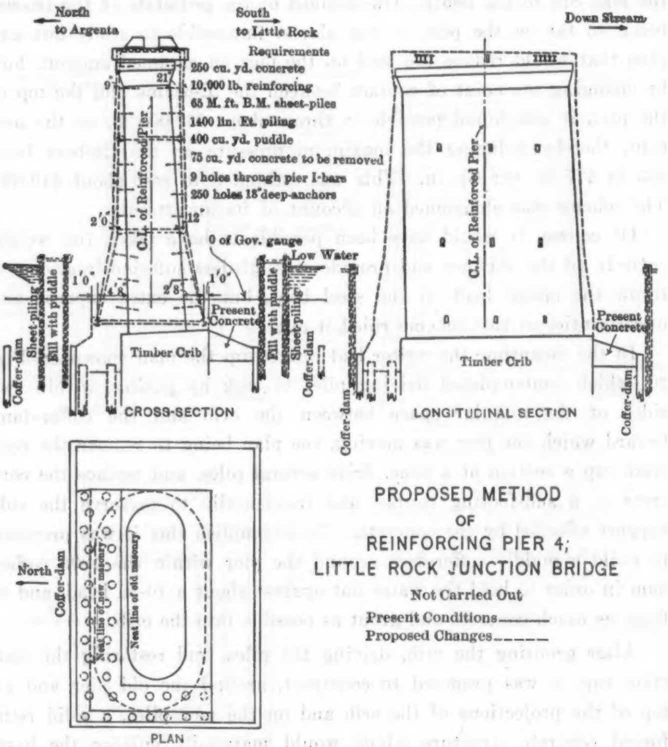


FIG. 20.

The writer has always felt that this method of repairing Pier 4 was the best that could have been devised, but so many objections were raised, as to the difficulty of extending the old coffer-dam up to make it water-tight, the danger to the pier of removing the concrete cap placed in 1899, the difficulty of driving the piles, and the possible damage to the pier by the adjacent pile-driving, the danger of

boring holes in the old crib for grouting (and possibly letting the sand run out), the impossibility of getting proper bond between the new and old work under traffic, the danger of disturbing any of the former conditions without carrying the spans on falsework, etc., *ad infinitum*, that the scheme was dropped.

Proposed Falsework for Supporting Spans.

Strange to say, several engineers who apprehended that if any work were done on the pier it would fall down, and desired the spans placed on falsework to save the steel spans in case of the collapse of the pier, proposed to support the spans in such a way that if the pier did fall it would knock the supports out from under at least one span and cause that to fall. When the writer pointed out this condition, he was requested to arrange for such temporary support of the spans resting on Pier 4 as would hold them up in case the pier should fall, the idea being to have plans for such temporary supports thoroughly understood by and in the hands of all concerned, with authority for the division forces of the Railway Company to erect the supports immediately on the further rapid movement of Pier 4.

FALSEWORK FOR SUPPORTING TWO SPANS OF LITTLE ROCK JUNCTION BRIDGE CONTEMPLATING THE REMOVAL OF PIER 4

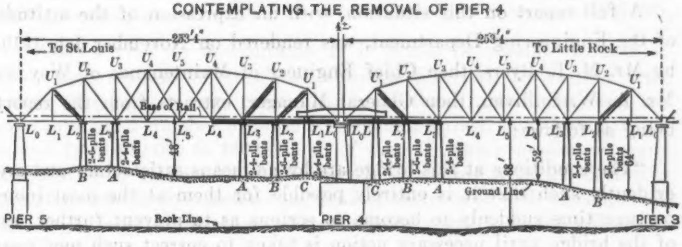


Fig. 21.

Fig. 21 shows the general plan that was worked up. On account of the necessity for retaining the greatest possible clear waterway, and the impossibility of maintaining any longitudinal bracing below high water except with disastrous results, it was proposed to place five double bents under each span, and to bridge over the pier with through girders, from which the floors would be hung. In the event of the unexpected happening at Pier 4, it was proposed to erect bents, *A*, under the third panel point on each side of Pier 4 and stiffen the tension members in the ends of the trusses so as to enable them to

resist the compression that would result if the pier were removed. It was expected that when the spans had been supported in this way, traffic could be suspended while the pier was being taken down to prevent its collapse, if the emergency should require; or, the spans having been protected against the collapse of the pier, traffic could be maintained until its collapse, and the remainder of the falsework could be put in in the meantime. The material for two double bents and for stiffening the overhanging ends of the trusses was assembled at the bridge and held there for several years, until after the reconstruction of Pier 4.

Further Studies.

Continuing into 1910, the entire matter was studied from every conceivable standpoint, including reconstruction of all the piers as single-track in their old location, reconstruction for double-track, an entirely new set of piers and shifting the spans, entirely new single- and double-track bridges, and rearrangement of the method of operation of the Little Rock Terminals so that the bridge could be abandoned, all traffic to use the main-line bridge. None of these plans was carried out, but the "watchful waiting" was continued.

A full report on this situation, with an expression of the attitude of the Engineering Department, was rendered on November 1st, 1910, by Mr. M. L. Byers, then Chief Engineer of Maintenance of Way, to Mr. A. W. Sullivan, then General Manager, extracts from the report being as follows:

"The conditions at this bridge are by no means satisfactory and are evidently such that it is entirely possible for them at the most inopportune time suddenly to become so serious as to prevent further use of the bridge until necessary action is taken to correct such new conditions. This may occur at a period of high water, which would prevent the use of the bridge for some six months to a year, although I do not believe there is serious danger of the loss of any of the superstructure of the bridge. It would probably be possible at any time (unless the failure were extremely sudden, which is unlikely) to drive such falsework as would be necessary to support the two spans resting on this pier, although in this case train service would have to be discontinued.

"There are a number of factors entering into the decision as to what should be done at this point. It may be that considerable time will be required to carry out the plans which may ultimately be de-

cided upon. It is impossible to determine how long the present conditions may continue. It may be that they will change almost at once, or it may be that they will not change for years. It would seem desirable, therefore, if possible, to decide upon a plan of action in case of sudden change for the worse so that no time will be lost in the preparation of plans or in necessary further study of the situation. To this end the following is submitted for your consideration:

"An entirely new set of piers for a single-track bridge, either in the present location or at one side of the present location, would cost approximately.................... $250 000.

"An entirely new set of piers for double-track bridge, either in the present location or to one side of the present location, using the present spans, would cost approximately.. $450 000.

"A new double-track bridge (including the piers) would cost approximately.. $800 000.

"The present Baring Cross Bridge (at *Y*, see Fig. 2) will probably require renewal within five years at an estimated cost of approximately (for double-track)........... $900 000.

"In the event of the failure of Pier 4 alone, the probable cost of entirely rebuilding it for single-track would be about .... $60 000.

"In the event of the failure of Pier 4 alone, the probable cost of entirely rebuilding it for double-track would be about .... $80 000.

"With a view to determining if it would be possible further to defer final renewal of this pier through resorting to further temporary expedient, a very careful study of the situation has been entered into and a number of tentative schemes worked out, these varying in cost from $12 000 to $50 000; all of them, however, present so many possibilities of casualty, and the cost as compared with the decidedly uncertain benefit is so great, that none of them meet with my approval, nor do I know of any temporary expedient which I am willing to recommend.

"As before stated the further period during which the present temporary expedient can be continued is entirely uncertain and there is the possibility that it may fail on extremely short notice.

"Figure 8 is a general elevation of Pier 4 and the two spans resting thereon and shows in red the temporary falsework necessary to be put in place in order to prevent these spans falling into the river in case Pier 4 falls.

"Figure 9 shows the character of the temporary piers and the two plans indicate the amount and character of the work which must be done in order to save the spans in case of the falling of Pier 4. It must be borne in mind that it is quite possible this work, if it becomes

necessary, must be carried out at the time of extreme high water, and it is absolutely impossible to foretell the amount of time that will elapse between the time when the rapid movement of Pier 4 is detected and the time when the pier will actually fall—we may have time to place this falsework and we may not. The falsework has been ordered and will be kept on hand at Argenta.

"Figure 8 shows in yellow the additional supports that must be provided in case of the loss of Pier 4 before the bridge can be again opened to traffic. During the period of high water there will, of course, be more or less amount of danger that drift or boats would destroy this falsework—such is the risk involved in the present conditions.

"Quite recently Pier 3 has begun to move longitudinally at a much faster rate than heretofore, and it is, of course, possible that the conditions with which we are confronted at Pier 4 may at any time have to be met in connection with movement of the other piers.

"Recommendations.

"In view of the varying circumstances above enumerated, it is my recommendation that it be decided at once what policy is ultimately to be followed in connection with this bridge. I believe all of the facts essential to this determination are stated in this report and the matter would seem to be one of policy rather than engineering.

"Yours truly,

"M. L. Byers,

"Chief Engineer, M. of Way."

Proposed Mat Around Pier 4.

For two or three years prior to 1911, the channel of the river had been moving away from Pier 4 and concentrating under the draw-span, resulting in the building up of the river bed around this pier with clear fine sand which early in 1911 covered the top of the crib and reached up on the masonry within 1 or 2 ft. of extreme low water. This sand evidently filtered into the crib and gave increased supporting power in addition to that provided by the sand that enclosed it. As a consequence, the movement of Pier 4 gradually slowed down, so that it was barely perceptible, and the great uneasiness that had prevailed for several years was partly allayed. The decrease in the movement was attributed to the steel bent placed at the back of the pier, by those responsible for that monstrosity.

In order to protect and preserve this sand surrounding the pier, the writer attempted in the fall of 1910 to secure authority to spend about $2 000 to construct a standard brush and pole mattress, about

150 ft. square, around Pier 4, to be securely bound together with wire and wire strand, and covered with rip-rap placed in pockets of the mattress to be formed by placing two sets of poles one across the other on top of the mattress to prevent the rip-rap from rolling off.

At the low stage of the river following the request for authority, the work could have been done without barges or other floating equipment and brought to completion in 30 days. The authority was not granted, and, after the next flood, it was not needed, as much of the sand had been washed away.

Rapid Movement of Pier 3.

While the river bed was building up around Pier 4, it was scouring away around Pier 3, the north rest pier of the draw-span, and this pier started to move quite rapidly directly up stream, at right angles to the axis of the bridge.

The movement in 1911 was more rapid and constant than any experienced previously in the other piers, and indicated the necessity for immediate action. The pier did not move dangerously in the direction of the bridge. Its natural tendency would have been to move south, as the plumb south face and eccentric bearings of the spans made the pressure heaviest on the south side. It was restrained from movement in that direction, however, by the rods and yokes which made this pier an anchor for the pull of Pier 4. In fact, during the frequent tightening of the yoke rods, Pier 3 was observed to move slightly north into a more erect position.

Plans for Reconstruction of Pier 3: Modjeski's Recommendation.

The first plan prepared for the reinforcement of Pier 3 was somewhat similar to the writer's last plan for Pier 4, which had been rejected. (See Fig. 20.)

It consisted of a reinforced concrete shell, tied to the old pier and resting on additional piles to be driven to rock around the old crib, as shown by Fig. 22. Here, however, there was no surrounding cofferdam, and it was deemed advisable to furnish protection for the piles and additional stability for the pier.

This was to be obtained by sinking around the pier mud-cell mattresses in layers about 4 ft. deep, with enough rip-rap in them to sink them. The result would be a building up of mattresses of

decreasing sizes and rock; sand sediment would immediately deposit until the top of the crib was nearly reached, when the entire mass would be covered by a heavy layer of rip-rap, the latter and the surrounding bed of the river would then be covered by another large protecting mattress. It was also the intention to bore holes in the crib and get in as much grout as possible. The writer believes this method of repair would have been entirely satisfactory; it also had the advantage of being very cheap. Five bids were received, the lowest (from an entirely reliable contractor) being about $8 500, which, together with the extras and incidentals, would probably have made the total cost of the work $10 000.

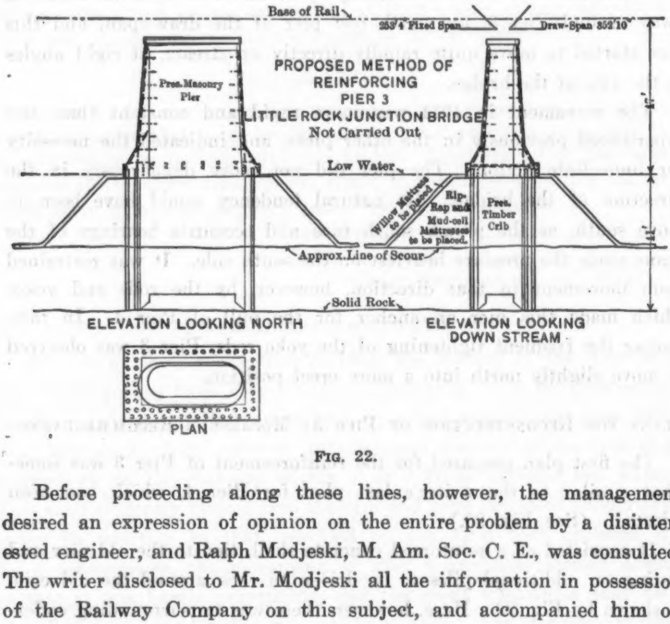


FIG. 22.

Before proceeding along these lines, however, the management desired an expression of opinion on the entire problem by a disinterested engineer, and Ralph Modjeski, M. Am. Soc. C. E., was consulted. The writer disclosed to Mr. Modjeski all the information in possession of the Railway Company on this subject, and accompanied him on an inspection of the bridge.

Copies of Mr. Modjeski's reports of August 5th, August 7th, and August 19th, 1911, are given in Appendix D.

It was arranged between Mr. Modjeski and the writer to bore holes down through the crib to rock, and this was done as far as

possible. It was found to be very difficult to get the pipe down; it would encounter solid timber, then sand, then loose rock, and combinations of all these. However, it was finally driven to the solid timber roof of the caisson, which was encountered at such an elevation as to leave no doubt that the cutting edge rested on rock, the level of the latter having been ascertained by running a jet pipe down through the sand around the pier. In sinking the pipe through the crib, efforts were made to secure the services of expert well drillers, but they all refused to guarantee to get the pipe down.

In brief, Mr. Modjeski recommended first that the sand in the crib of Pier 3 be removed and the crib filled with concrete. To accomplish that, he recommended driving 50-ft. triple-lap sheet-piling entirely around the pier, 6 ft. from the crib, and examining the old crib by partly removing the old planking. He also recommended that the up-stream end of Pier 3 be supported on shores resting on piles driven alongside the old cribs. This work involved supporting the spans on falsework.

A later report suggested placing the coffer-dam 12 in. from the old crib (if it was found that the filling of the cribs was sand which could be pumped out), excavating the material between the old crib and the coffer-dam, and sealing the bottom with concrete. Then the top crib timbers and the material inside were to be removed and the crib refilled with concrete, this work necessitating the support of the spans on falsework and taking down the old pier.

Later, he recommended rebuilding Piers 3 and 4 entirely by supporting the spans on falsework, putting coffer-dams around the piers, taking down the masonry, filling the cribs with concrete, and building new piers on them.

After considerable study of these latest recommendations, it was tentatively decided to follow Mr. Modjeski's first recommendation, but, on account of the great head against which pumping would have to be done (up to 50 ft.), steel sheet-piling was contemplated. Detailed estimates, however, indicated that this method would be quite expensive and uncertain, and would involve the hazard of carrying the spans on falsework. As a slight August rise in this river, a few days before, had swept away 1 000 lin. ft. of falsework at a bridge about to be erected farther up stream, those responsible for this bridge were not enthusiastic about carrying the spans on falsework.

Taking down the piers to rebuild them entirely would have been fully as hazardous, and very expensive.

Mr. Modjeski was then requested to pass opinion on the writer's plan of driving piling around Pier 3 and resting on it a reinforced concrete encasement around the pier, protecting the piling with mud-cell mattresses and rip-rap, as shown by Fig. 22. This he objected to as somewhat uncertain as to results and the eventual stability of the pier.

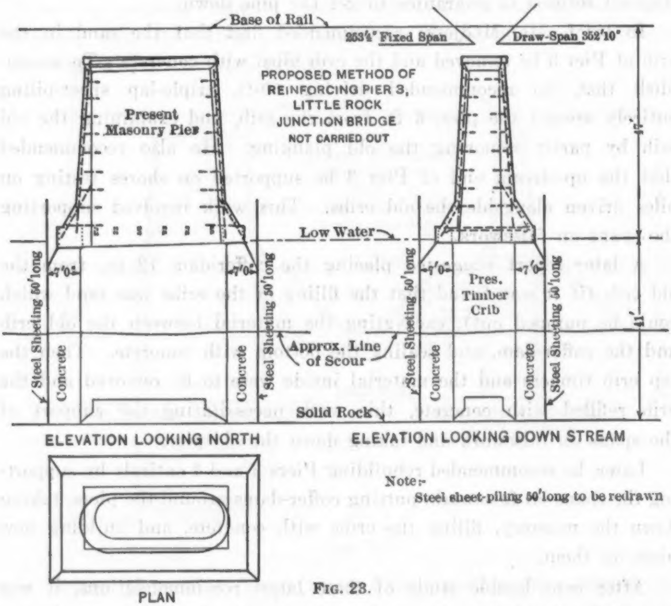


FIG. 23.

He was also asked to pass on a plan for driving steel sheet-piling about 6 ft. from the old crib, excavating the space between it and the old crib, and filling that space and the crib with concrete, the pier to be encased in a reinforced concrete shell, the whole to be done without falsework. Fig. 23 shows this scheme. He pronounced it to be very expensive, and said:

"To attempt to carry out this scheme without supporting the spans on falsework would seem to me extremely risky. The excavation around the crib would deprive it of considerable skin friction and would

probably open up more places through which sand now in the crib could leak out. The crib is now settling at the rate of 1 in. per month by the crushing of the timbers and this settlement may under the new conditions get so rapid as to endanger the whole structure.

"I am convinced that the pier could be built new for less money than the schemes shown on your plan. The new pier would probably contain about 500 cu. yd. of concrete and there may be another 500 or so necessary to fill the crib, making, say, 1 000 cu. yd. of concrete in all, besides, the work would be more certain of accomplishment and with the least amount of unforeseen expensive contingencies. I would therefore strongly recommend that this pier be rebuilt as stated in my former letter to Mr. Pearson. While it is very difficult to make even an approximate estimate on the cost of such work, my opinion would be that the new pier would cost very little more, if any, than the cheaper one of the two schemes in question, assuming in all cases that it would be desirable to support the adjacent spans, as there is great uncertainty how the present crib would behave while any work is being done around it.

"To rebuild the pier I would proceed as follows: Build temporary supports for the span, carrying the ends on a set of girders. Take down the old pier to the crib work. Drive wooden triple-lap sheet-piling or steel sheet-piling around the present crib. Excavate present crib and fill with concrete by methods which will become more apparent as the work goes along. Build the new concrete pier on top of the crib work thus reinforced."

The method of reinforcement that was finally brought to successful conclusion was first suggested by E. J. Pearson, M. Am. Soc. C. E., First Vice-President of the Railway Company. To overcome all uncertainties, he suggested securing the necessary additional supporting power by the use of a pneumatic caisson on each side of the old crib. This was extended, later, by adding caissons for the ends, forming an annular caisson surmounted by a coffer-dam, the space above the annular caisson to be filled with concrete, doing the same to the crib, and encasing the old pier in a new reinforced concrete shell, as shown by Fig. 24. This was also referred to Mr. Modjeski by the writer, in a letter setting forth the plan, as follows:

"I have never fully abandoned the idea which you first proposed of filling the pockets within the crib with concrete, and think if this work can be accomplished that the problem will be solved. I find upon investigation that it will cost very little more to sink an annular pneumatic caisson entirely around the crib than to carry out the scheme formerly proposed to use steel sheet-piling and concrete filling.

"I hand you herewith blue print, showing in red the construction of the caisson. I would construct the working chamber entirely of steel, as this can be done, by the use of second-hand bridge members now on hand at our store yard, for very little cost. I would leave off the inside wall of the coffer-dam above the roof of the working chamber, and construct a water-tight outer casing wall braced by latticed steel members extending up to low water.

"After sealing this working chamber with concrete, this outer shell would form a coffer-dam entirely around the pier.

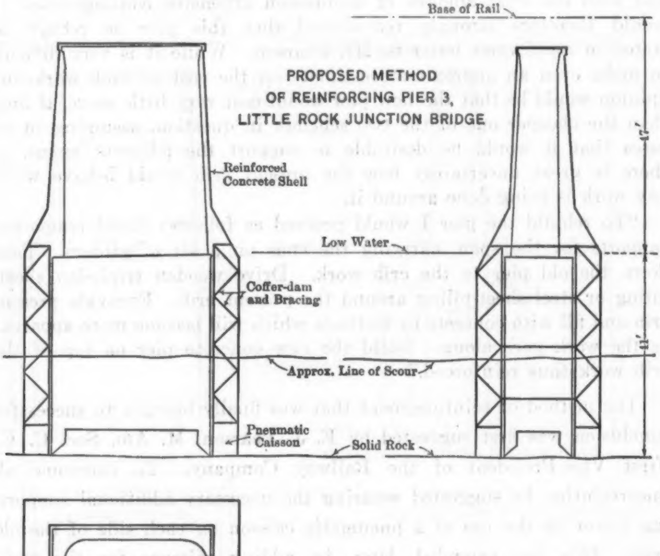
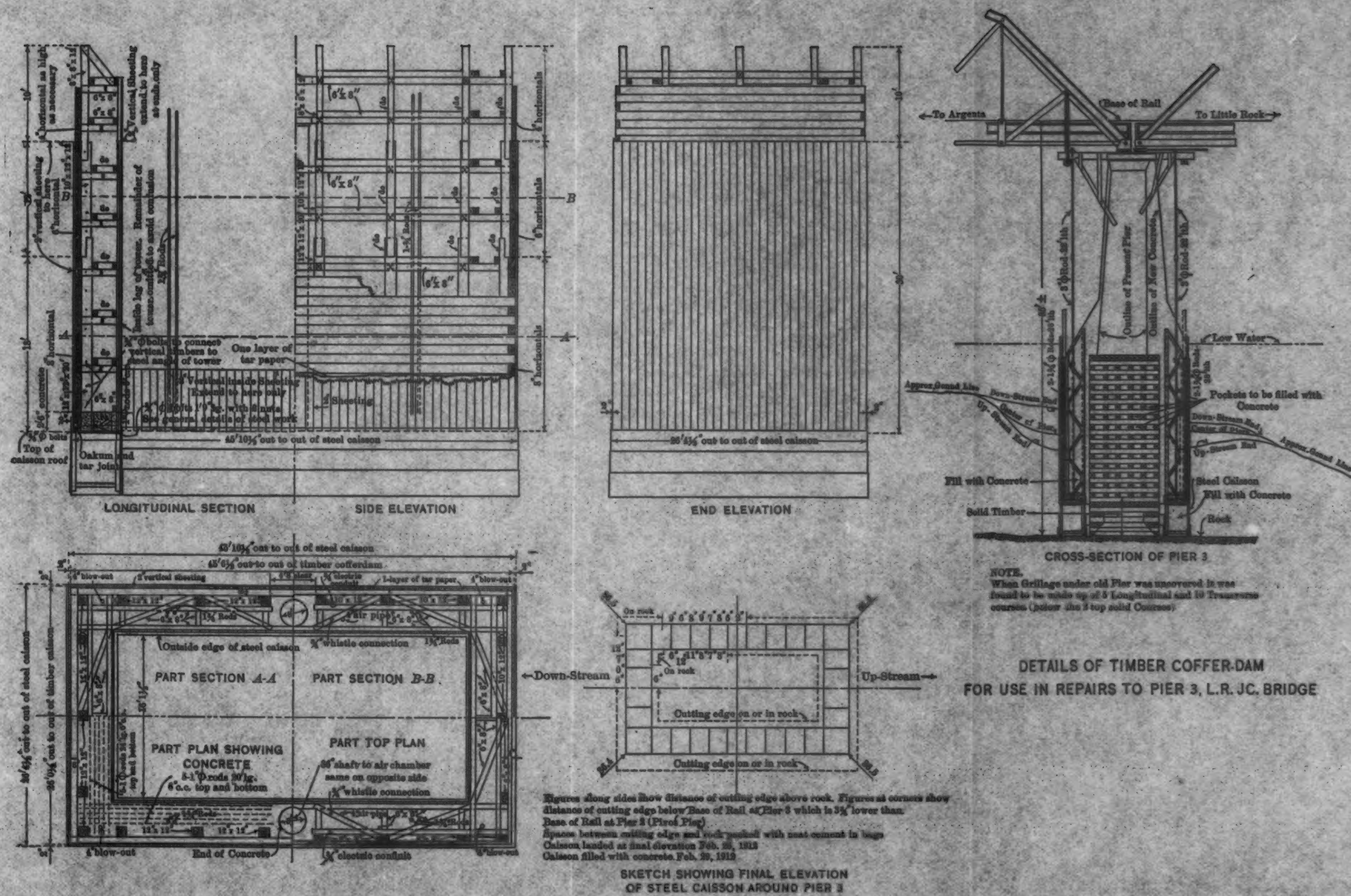


FIG. 24.

"The Arkansas River is subject to very rapid rise and excessive scour, and at such times carries an immense amount of drift. It is a hazardous proceeding at any time to carry spans on falsework in this river. During the construction of our Baring Cross Bridge, I understand, one of the spans was lost on this account. The Arkansas Bridge Company recently lost 1 000 lin. ft. of falsework in this same river a few miles above Little Rock.

"For this reason I do not like to place spans on falsework and take down the pier, except as a last resort.

PLATE III.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXIX, No. 1335.
SMITH ON
REPAIRING BRIDGE PIERS.



DETAILS OF TIMBER COFFER-DAM
FOR USE IN REPAIRS TO PIER 3, L.R. JC. BRIDGE

"If the pier starts to show signs of more rapid movement during the sinking of the caisson, traffic could be removed from the bridge temporarily and the sinking proceeded with. The pier could then be supported by temporary struts and shores resting on the steel members above the working chamber. After the caisson were sunk to the full depth and the working chamber sealed, I would pump out around the present crib and ram the pockets full of concrete, using a cement gun for the purpose, if necessary, and working from both sides of the crib. I would then complete the work by completely filling the space above the working chamber with concrete, and would then encase the present masonry pier with reinforced concrete above low water, both for the purpose of securing a symmetrical appearance and also to distribute a considerable portion of the load to the caisson."

Mr. Modjeski thought that this plan would be unduly expensive and, if carried out without falsework, entirely too hazardous. As he did not disapprove the scheme, however, it was adopted and carried out successfully.

Reinforcement of Pier 3 in 1912.

Unfortunately, the delay in arriving at a decision as to what would be done at Pier 3 consumed the best months of the year for the work. The change to pneumatic construction, necessitating the assembling and erection of the pressure plant and the construction of the caisson, further delayed the work up to the time of the flood period. The compressor plant was finally ready, and the steel caisson was shipped early in November.

Plate III shows the general outlines of the annular caisson and coffer-dam. The old records showed the old caisson under Pier 3 to be 14 by 31 ft., so the dimensions of the inner cutting edges of the new caisson were made 16 by 33 ft. The caisson was made of steel primarily to reduce its necessary width and consequently the displacement; the presence in the old bridge-material yard of a large number of duplicate second-hand floor-beams which could readily be converted into a caisson also encouraged the use of steel. The details of the steel caisson and the joint between its roof and the coffer-dam are shown on Plate IV. It was decided to build up a coffer-dam on the outer wall of the caisson, connecting the coffer-dam to steel lattice work or towers bolted to the roof of the caisson. These towers were provided in order to form the backbone of the rigid framework it was known

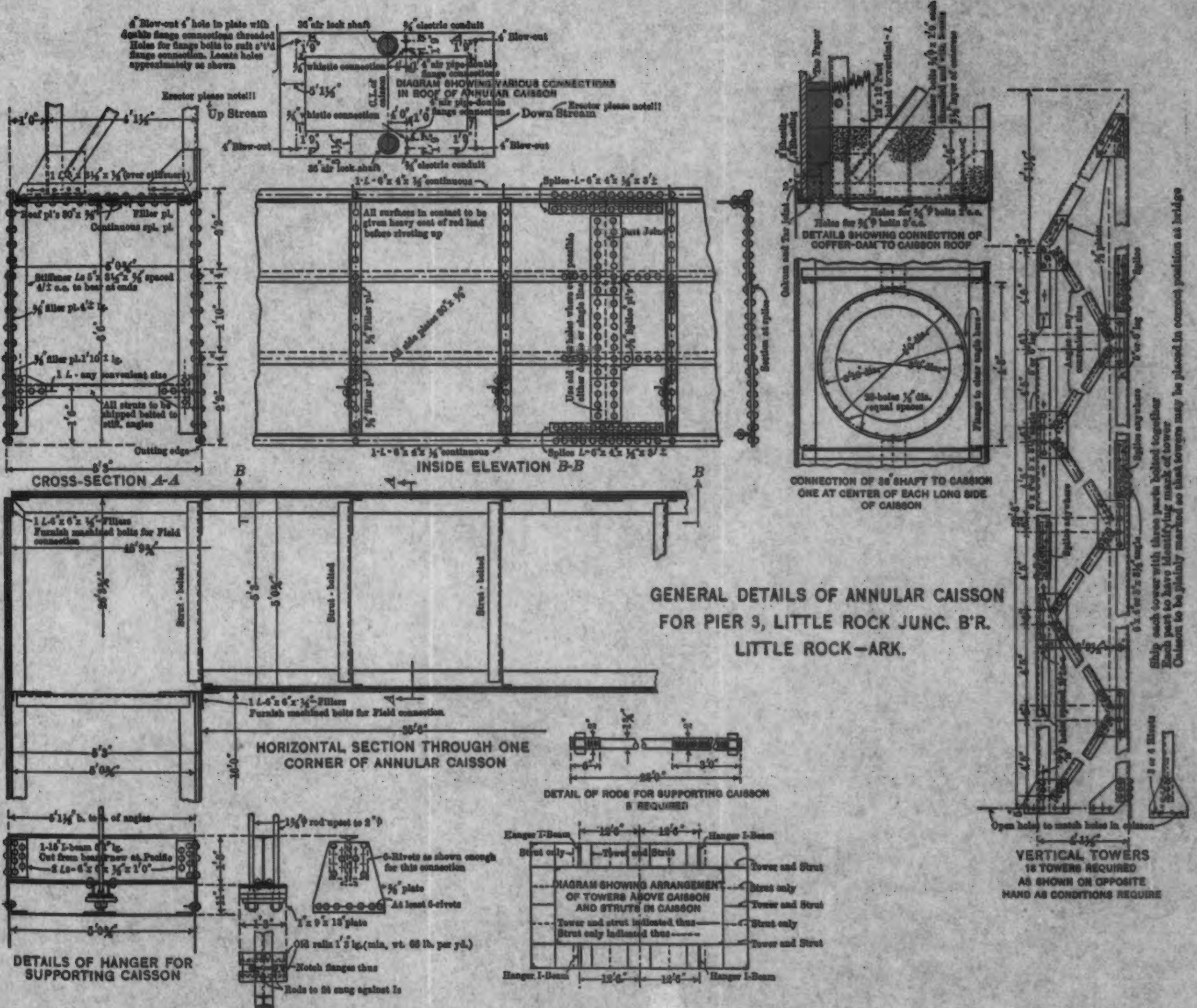
would be required to brace the new coffer-dam against the old crib during excavation.

The greatest care was taken in the design of the joint between the caisson and the coffer-dam. The timbers in the bottom row, 8 in. thick, were cut to fit the steel as closely as possible, and were set in a bed of hot tar and oakum. Surfaced timbers were used, and each layer was set on the next below in hot tar and caulked. Tar-paper was then placed over the timbers and 2-in. matched sheeting placed outside. The lower ends of the sheeting were driven into a groove, filled with hot tar and oakum, over the roof of the caisson, formed between the bottom timber and the upstanding leg of the angle at the top corner of the caisson. A space about ½ in. wide, between the sheeting and the angle, was then caulked. To bind all parts together effectually and to provide additional weight for sinking, a 2 ft. 6-in. layer of concrete was then placed over the roof of the caisson enclosing the bottoms of the towers. This concrete was banded to the steel roof by a large number of ¾-in. bolts, 21 in. long, fastened to the steel plate and extending vertically into the concrete.

The construction of the remainder of the coffer-dam was similar to that just described.

For the support of the annular caisson during the assembling and sinking, until the entire load rested on the river bed, the temporary support from the pier and trusses shown on Plate V was provided. Two 8 by 16 in. by 28-ft. timbers were placed across each end of the pier, as close as possible to the truss shoes. The coping stones at each end had to be removed in order to place the timbers low enough to permit the draw-span to swing over them. The ends of the timbers were held up by inclined struts gained into their under sides and set in grouted notches cut into the masonry of the pier. For additional stability and stiffness, the north end was connected by rods and struts to the end posts of the north span which, in turn, were stiffened by timber struts (similar to collision struts) running down to the first panel points. On account of the necessity of having the draw-span free to swing, it was not possible to provide corresponding connection with the end post on the south side of the pier.

Four 3-in. steel rods, threaded for their entire length, were passed through between the ends of the timbers, the nuts on them resting on heavy plates over the timbers, and the lower ends of the rods



GENERAL DETAILS OF ANNULAR CAISSON
FOR PIER 3, LITTLE ROCK JUNC. B'R.
LITTLE ROCK—ARK.

reaching down to chairs made of rails supporting other smaller rods in pairs, on which the long sides of the caissons were supported near the water level after being floated out to the pier. The short sides or ends were then floated on barges and connected to the long sides by machine bolts. After the caisson was assembled and caulked, and all shaft, pipe, and wire connections were made, it was lowered into the water and the construction of the coffer-dam was started.

During and after the placing of the 30-in. layer of reinforced concrete over the roof of the caisson, the sinking progressed as fast as the work permitted. At the commencement of sinking, the river bed at the down-stream end of the caisson was several feet higher than at the up-stream end. As the caisson was lowered, the sand scoured deeper and deeper at the up-stream end and the workmen shoveled the sand from the down-stream end of the caisson into the water at the up-stream end until the cutting edge landed all around, following which the sand and other material, much of which consisted of heavy rip-rap and logs (almost petrified), was removed in buckets through the locks. The removal of the sand formerly against the down-stream end of the pier relieved it of pressure to such an extent that it partly righted itself. Trouble was continually experienced by the caisson catching on the protruding upper ends of the sheeting planks of the old crib, which had been torn loose by the settlement of the timbers and bulged by the pressure of the material escaping from the crib. These were particularly annoying before the cutting edge rested on the river bed, as the workmen had to pick their footing in the caisson. On all the old records available the caisson of the old pier was shown to be plumb, and a clearance of 2 ft. was supposed to have been allowed between it and the new caisson, but it was found that the old caisson had a sharp batter, which necessitated ripping off the old vertical sheeting before the new caisson could be landed. The cutting edge of the old pier was found on rock everywhere except at short dips which had been filled with concrete.

All the material within the new caisson had to be removed through the locks because sufficient pressure could not be maintained to force the sand through the blow pipes. This was caused by all the material between the old and new work running into the caisson and leaving the inner cutting edges usually exposed; and, when an attempt was made to increase the pressure to blow out the material,

the air would blow out under the inner cutting edge instead. As it was desired to avoid the resulting washing out of sand from the old crib caused by the commotion in the water during these blow-outs, the efforts to use the discharge pipes were abandoned. Mattson locks were used, and permitted the rapid removal of the material. No other particular difficulties were encountered.

The cutting edges were everywhere landed on rock, except over short dips, which were cleaned out and filled with concrete. The working chamber was filled with concrete, carefully rammed, to the roof, and before it had set, the lower sections of the shafting were also filled with grout.

Fig. 25 shows the steel caisson after it had been lowered partly into the water and construction of the coffer-dam had commenced. Fig. 26 shows the upper edge of the coffer-dam after the caisson had been landed and the reinforcing rods for the concrete encasement were being placed.

While the caisson was supported from the pier the inspector was required to compute daily and report weekly the weight supported. To guide him in the control of the work, and to avoid overloading the temporary supports, he was told the maximum weight that could be allowed, and was given a copy of a typical programme of work and a tabular summary of weights, together with the following instructions, the whole idea being to control the construction and sinking in such a manner as to keep the smallest load on the rods and take advantage of the immersion:

"Instructions.

"The support that has been provided for lowering the caisson into its final position is capable of safely supporting a total load of 325 000 lb., and the total weight of the caisson and coffer-dam and other materials above the caisson suspended from the rods should not exceed the above weight.

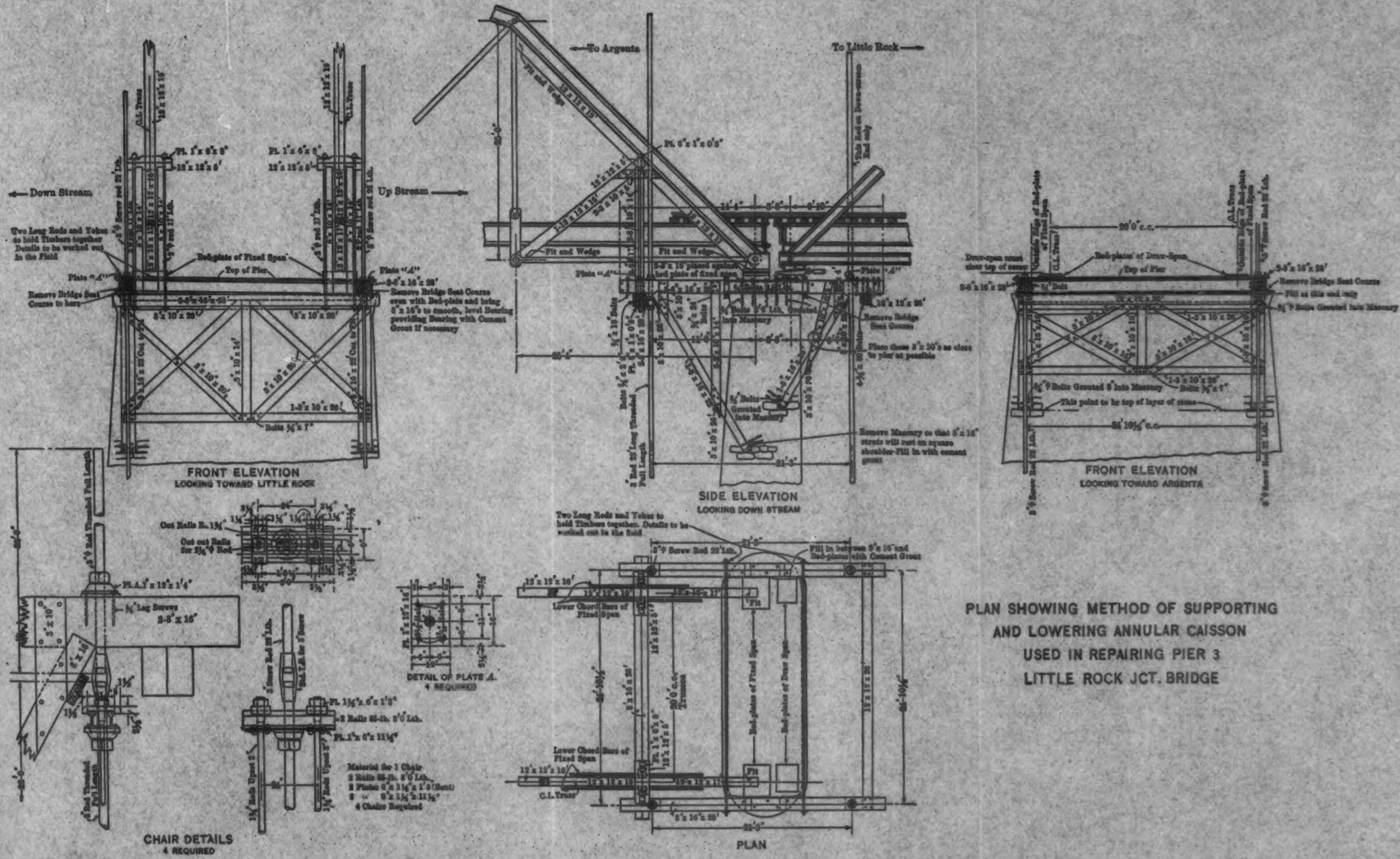
"There are five conditions that must be met in the sinking and founding of this caisson.

"They are as follows:

"1. Total weight of structure before lowering into water must not exceed the strength of the supporting rods.

"2. Total weight of structure decreased by the buoyant effect of the water displaced during sinking must not exceed the total supporting power of the rods.

PLATE V.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXIX, No. 1385.
SMITH ON
REPAIRING BRIDGE PIERS.



PLAN SHOWING METHOD OF SUPPORTING AND LOWERING ANNULAR CAISSON USED IN REPAIRING PIER 3 LITTLE ROCK JCT. BRIDGE

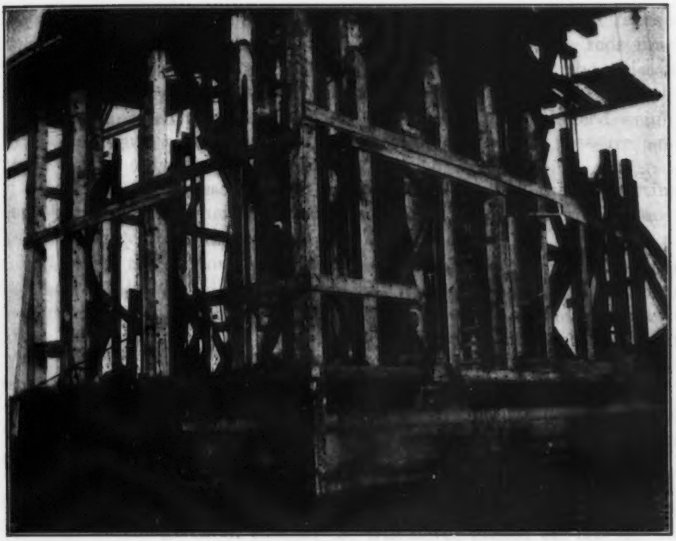


FIG. 25.—LOWERING COFFER-DAM AROUND PIER 3.

FIG. 26.—STEEL CAISSON AND COFFER-DAM AROUND PIER 3.

"3. In order that the caisson may be filled with air when it reaches the bed of the river, the total weight suspended from the rods must be approximately equal to the weight of water within the caisson, which will be expelled by the application of the air.

"It is expected that when the air is applied the unbalanced weight of the structure will be practically supported by the air pressure, and there will be only a nominal load on the rods.

"4. After the caisson lands on the sand, so that the supporting rods become slack when the nuts are turned, additional weight must be provided to force the caisson into the ground.

"5. After the caisson has reached bed-rock and the working chamber has been sealed by filling it completely full of concrete, there must be sufficient total weight to hold down the structure and prevent it from rising, on account of water pressure caused by the head of water from the surface of the rock to water surface, to be assumed acting over the entire base of the caisson.

"The attached table has been worked out in detail to show the various steps that should be taken in the construction of this caisson and coffer-dam.

"It is probable, of course, that you will not follow the exact procedure outlined, but you should follow it sufficiently close, so that the total weight given in column 'N' will never exceed 325 000 lb.

"Please note that the weights given in the table are assumed weights, and there has been omitted the weight of men, tools, lines, and other small equipment that will be supported upon or within the coffer-dam during sinking; the weight of such material must be added to the total weight.

"In the preparation of this table, the actual weight of the steel caisson, the towers, and the air shafts and locks have been used. The weights for timber, concrete, and iron, such as drift-bolts, through-bolts, spikes, and other small hardware have been assumed.

"Please arrange to weigh sufficient of the timber that will be used in this work and estimate its weight per cubic foot so that the latter figure can be used in your accurate calculation of weights.

"In addition please arrange to make up and weigh several cubic feet of the concrete that will be used, in order that its weight per cubic foot may also be known for use in the calculations. The concrete should be weighed when full of water as it will always be filled with water when submerged.

"I also hand you a number of white prints in blank, on which please arrange to enter notes and figures similar to those shown on the typical summary handed you herewith and arrange to send me one of these weekly with copy direct to Bridge Engineer, C. E. Smith.

"Please understand that this table must be posted up to date every day, and you must instruct your Inspector at the bridge to have

it always in shape for inspection and approval by the Bridge Engineer at all times, as the Bridge Engineer will inspect it at such times as he may visit the bridge.

"Please understand that an increase of the loads over the weights given you above would be very disastrous, as it would undoubtedly result in the breaking of some portion of the supporting structure, and might have a disastrous effect upon Pier 3.

"Please note by reference to the typical summary that the application of air to the caisson will make the structure practically self-supporting, but it is not safe to assume that the structure can be supported by the caisson full of air, as a leak in the caisson or in the supply pipe (which is very long), or a failure in the air compressor, or reservoir, or boilers, or a blow-out under the cutting edge, would immediately decrease the supporting power and increase the load in such a way as to throw a very heavy stress upon the rods, which might result in their failure.

"The caisson must be carefully adjusted level when it is first placed, and must at all times thereafter be kept perfectly level as the lowering of one corner faster than any of the others will result in racking the caisson and coffer-dam to such an extent that heavy leaks may be expected. It will be a very easy matter to keep this structure perfectly level by proper manipulation of the screws.

"In case one corner or one portion of the cutting edge rests upon an obstruction, the obstruction must be removed before further sinking is done. After the caisson reaches a bearing upon the bed of the river and the air has been applied, the structure will, to all intents and purposes, be floating, and it will be necessary to supply additional weight to force it down through the sand. This weight will be provided by placing concrete in the end compartments under water.

"The plans call for these compartments to be supplied with 2-in. forms for concrete for their entire height, so that, if necessary, the entire height of the structure can be built up at the ends to provide this additional weight. In addition it is desired that temporary 2-in. sheeting be placed on the inside of the long sides to prevent sand and other material running out of the old crib on top of the concrete. In no case, however, must any concrete be placed in the long sides, as it is desired that those spaces be left open until after the old crib is repaired.

"It will be impossible to add the weight in the end pockets while the structure is swinging from the rods, as it would not be practicable to provide a support of sufficient strength to support such a very heavy load.

"It is impossible to say how much concrete must be added in these end compartments to force the caisson through the material in

the bed of the river, as that resistance cannot be calculated, but the conditions must be conquered when encountered.

"After the caisson has reached bed-rock and the working chamber has been filled with concrete, the end pockets will be filled with concrete to a height of 25 ft. above the reinforced concrete roof, part of this material being placed during sinking through the sand and the remainder after the caisson has been sealed."

The weight of the steel caisson, the coffer-dam surmounting it, and the layer of concrete over the roof were not sufficient to sink the caisson after the river bed took the load off the rods. Inner forms for concrete in the ends of the coffer-dam were built up as the caisson was lowered, and concrete was deposited in them with bottom-dump buckets as the weight became necessary. Rods were placed in this concrete projecting from the inner face so as to form a bond with the remainder of the concrete to be placed later.

While waiting for the concrete in the working chamber to set, preparatory to pumping out the coffer-dam, a rapid rise came down the river and overtopped the dam on February 29th, 1912. The water subsided in about 6 weeks, and when pumps were started it was found that the coffer-dam had acted as a settling basin and was full nearly to the top with fine packed sand. The removal of this sand proved very burdensome, slow, and expensive, especially on account of the presence of the numerous braces. In the face of another rise, the dam was extended up to 52 ft. above the caisson, but the water went away over that again several times and stayed up for a considerable period, depositing more sand within the dam. The removal of this material, however, presented no difficulty other than the cost and delay. One pump easily controlled the leakage up to a 50-ft. head, giving a dry dam within which to work on the old pier. The filling of the old pier was found to consist of some rip-rap and more sand. In order to remove the sand, much rock had to be taken out. In fact, all the loose rock was removed with the sand, and only the tight rock was permitted to remain. The latter and the entire inside of the crib was washed out with a strong jet and then filled carefully with concrete. On account of the rather rapid settlement of the pier up stream, that end of the crib was first concreted, and, following the placing of that stiffener, the squeezing of the timbers at the down-stream end caused the pier to settle in the other direction, righting itself several inches. The entire space inside the coffer-dam

was filled with concrete, being brought up as the concreting of the crib progressed.

After filling the crib and the surrounding space with concrete, the pier was safe, but, as it was canted so much that it presented a poor appearance, and as the draw-span had only a very small bearing on the pier and was causing the upper courses of masonry to break away, the pier was encased in a reinforced concrete shell. As the new caisson had made the footing large enough, the reinforcement was built for double-track. Plate VI shows the details of the reinforced pier.

The tie-bars at the bottom were placed through bolts cut in the pier, and grouted in. The bridge seats under the spans were entirely renewed in concrete finished at the proper level for double-track spans. The difference in level was made up by concrete pedestals placed in sections, as shown by Fig. 27.

No falsework was used for the support of either span, and the bridge was never out of service during the conduct of the work.

The outer surface of the timbers in the crib under Pier 3 were squeezed down at the ends an almost imperceptible amount. The upper timber was burst open on account of the crushing load.

Fig. 18 is a view of Pier 3 after the completion of the work. The equipment used in this work is listed in Appendix E.

Reconstruction of Pier 4 in 1912.

In anticipation of further work to be done at Pier 4, and in recognition of the fact that it was in much worse condition than Pier 3, it was decided to replace the rods and yokes holding Piers 3 and 4 to the intermediate span by a pair of cables attached in such a manner as to relieve the spans from the pull. As an anchor for each cable, two 4 by 1-in. eye-bars, together with substantial anchors, as shown by Fig. 28, were embedded in the new concrete of Pier 3 near the top just beyond the ends of the old pier.

Before proceeding with work at Pier 4, two 1¾-in. steel cables with cast-steel clevises were attached to the anchor-rods at Pier 3 and suspended from the span. At Pier 4 two 24-in. I-beams were laid on flat, north of the pier, and placed in contact with two 60-ton jacks which were placed horizontally with their bases securely braced against the face of the pier. Tension was put in the cables by the jacks, and the

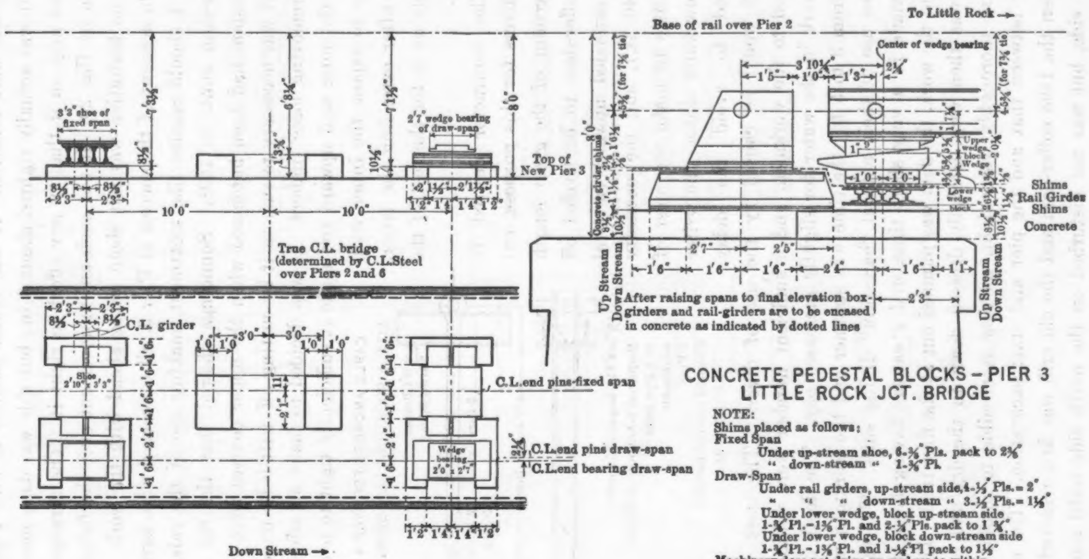
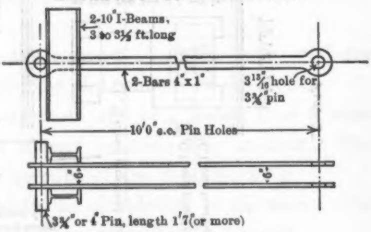

To Little Rock →
Base of rail over Pier 2
3'3" shoe of fixed span
2'7" wedge bearing of draw-span
Top of New Pier 3
Center of wedge bearing
Upper wedge block
Wedge
Lower wedge block
Concrete girder shims
Shims
Rail Girdes
Shims
Concrete
True C.L. bridge (determined by C.L.Steel over Piers 2 and 6
C.L. girder
Shoe 2'10" x 3'3"
C.L.end pins-fixed span
Wedge bearing 2'0" x 2'7"
C.L.end pins draw-span
C.L.end bearing draw-span
Down Stream →
Up Stream
Down Stream
After raising spans to final elevation box-girders and rail-girders are to be encased in concrete as indicated by dotted lines
CONCRETE PEDESTAL BLOCKS – PIER 3
LITTLE ROCK JCT. BRIDGE
NOTE:
Shims placed as follows:
Fixed Span
Under up-stream shoe, 6-3/8" Pls. pack to 2 3/8"
" down-stream " 1-3/8" Pl.
Draw-Span
Under rail girders, up-stream side, 4-1/2" Pls. = 2"
" " " down-stream " 3-1/2" Pls. = 1 1/2"
Under lower wedge, block up-stream side
1-3/4" Pl. – 1 3/8" Pl. and 2-1/4" Pls. pack to 1 3/4"
Under lower wedge, block down-stream side
1-3/4" Pl. – 1 3/8" Pl. and 1-1/4" Pl pack to 1 1/2"
Machinery does not drive up wedges to within about 4" of full entrance up-stream side and 6" down-stream side.

FIG. 27.

old yokes were removed. It was found possible to kick Pier 4 over and it was actually righted about 6 in., but this was done more to get a good grip on it than in any effort to right it. The arrangement is shown by Fig. 29. It was such an improvement over the former yokes that no hesitation was felt about going ahead with the work.

On account of the success at Pier 3, all concerned were agreed that Pier 4 should receive like treatment, making use of the caisson and coffer-dam sunk in 1898. Soundings showed that the top of that coffer-dam had been butchered when the upper courses were removed, and it was necessary to smooth it by cutting off drift-bolts under water and removing loose timbers. It was decided to make an effort to extend the outer wall upward and make it sufficiently tight to permit the pump to expose the inner wall, and, after extending the latter, to rely on it for the rest of the work.

CABLE ANCHORS FOR PIER 4
LITTLE ROCK JUNCTION BRIDGE



Required 2-Anchors as Shown
or 4-Eye-Bars
4-I-Beams
2-Pins
Cleaned but not painted

FIG. 28.

The contractor desired to use divers for this purpose, but on account of the great length of coffer-dam to be leveled off and extended upward (a total of 240 ft.), and the necessity for haste to make the most of the low-water season, the writer desired to avoid the delay which would be caused by the use of divers. The work was explained to the workmen, and a sufficient number were willing to do the work in the water for slightly increased pay. The depth of water was from 3 to 5 ft.; the time was summer and the workmen made extensive use of their natural "bare-skin" diving suits. The extension was made in a very short time, and a canvas having been stretched around the work, the dam was pumped out at the first trial. The inner wall was then very carefully dressed down in the dry and extended upward thoroughly water-tight, making it possible to pump out the water between that and the pier and start excavation. The concrete between the 1898 coffer-dam and the old crib was first removed on the south side, but was not disturbed on the north side until some time

PLATE VI.
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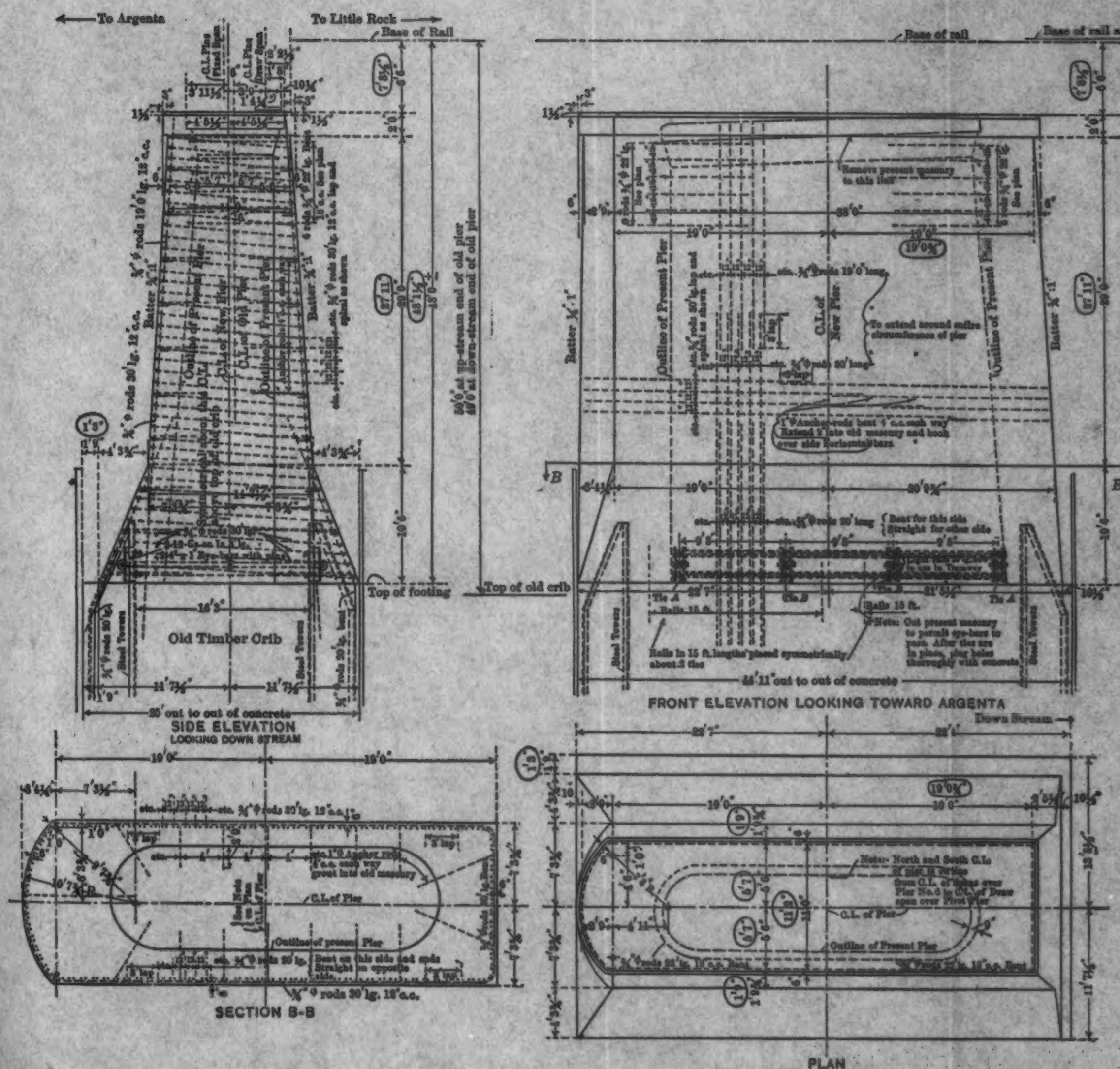
SMITH ON
REPAIRING BRIDGE PIERS.



DETAILED MASONRY PLAN FOR RECONSTRUCTION OF PIER 3 LITTLE ROCK JUNCTION BRIDGE

NOTE:-
New work shown thus
Old work to be removed shown thus
Old work to remain in place
Use 1:2:4 Concrete
Rods not to come nearer than 3" to face of concrete
Clean and moisten thoroughly old masonry where joined by new concrete.
Lap all rods 3'0"
Anchor-rods, 1" Rd., must be grouted thoroughly into old masonry and hooked over outside horizontal bars thus
Bevel 1" all corners 90° or less

Figures enclosed thus (7' 8¾") show pier as built.
Top of old pier projects above top of new pier 5" at up-stream end and 9" at down-stream end.

later, after the yokes and cables were in place anchoring the pier to Pier 3.

The removal of the material down to rock was slow and tedious. The old coffer-dam had been badly racked during sinking, and large leaks were found all the way down. They were overcome, and the rock was laid bare and covered with concrete up to the top of the caisson of the old pier, after which the crib was cleaned out, filled with concrete, and surrounded as at Pier 3.

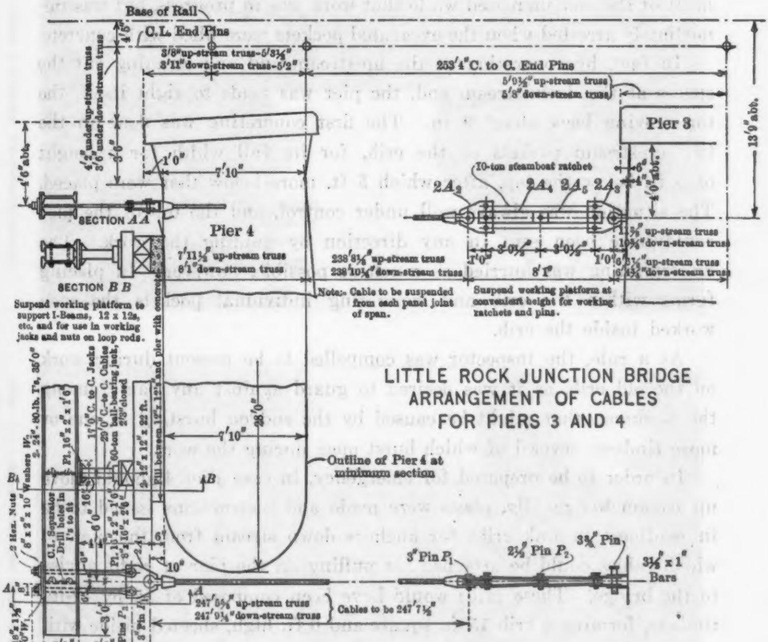


FIG. 29.

There was always a great flow of water from the south side, and in excavating down it was found that it came through under the cutting edges of the 1898 coffer-dam where a dip in the rock had been bridged over and was supposed to have been filled with concrete, but apparently was not. Several bad blows, carrying many yards of sand and flooding the excavation, occurred under the 1898 caisson, and it

was not possible to pump down to the bottom until after the material had been removed and the leak stopped by placing a sealing course of concrete under water.

Some of the timbers were crushed to 6 in. in height, and on account of the inclination of the pier new cracks developed in the timbers while the excavation progressed. The weakest portions of this crib were concreted as they were reached. Attention was given to the up-stream end, and excavation was first made there. The movement of the pier increased while that work was in progress, but was immediately arrested when the excavated pockets were filled with concrete.

In fact, by concreting at the up-stream end and cleaning out the spaces at the down-stream end, the pier was made to right itself, the top moving back about 9 in. The first concreting was done in the two up-stream pockets of the crib, for its full width for a height of 5 ft. below the top, after which 5 ft. more below that were placed. The situation was always well under control, and the top of the pier could have been sent in any direction by guiding the work. The underpinning was hurried as much as possible, however. In placing forms within the crib and concreting individual pockets the men worked inside the crib.

As a rule, the inspector was compelled to be present during work on the old crib, as it was desired to guard against any panic among the workmen that might be caused by the sudden bursting of one or more timbers, several of which burst open during the work.

In order to be prepared for emergency, in case Pier 4 should move up stream too rapidly, plans were made and instructions issued to be in readiness to sink cribs for anchors down stream from the pier, to which cables could be attached for pulling on the pier at right angles to the bridge. These cribs would have been composed of 12 by 12-in. timbers, forming a crib 15 ft. square and 6 ft. high, sheeted inside with 3-in. boards 6 in. apart. When such a crib is weighted down with rock and dropped on the bed of the Arkansas River it becomes buried immediately and forms an almost immovable anchor. It was not found necessary to sink such cribs, however.

No falsework was used while Pier 4 was under reconstruction; the pier carried traffic at all times.

Figs. 30 and 31 are two views of Pier 4 during the reconstruction. The old timber yoke and rods connecting to the trusses, placed in

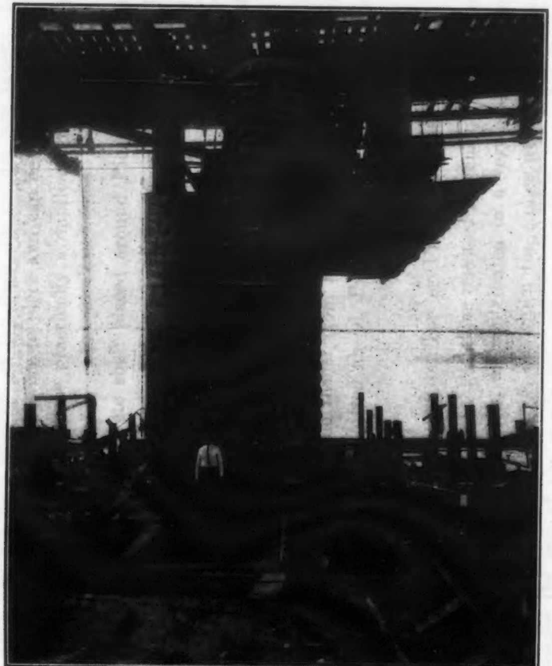


FIG. 30.—END VIEW OF PIER 4 DURING RECONSTRUCTION.

FIG. 31.—SIDE VIEW OF PIER 4 DURING RECONSTRUCTION.

FIG. 31.—Side View of Pier 4 during Reconstruction.

FIG. 30.—End View of Pier 4 during Reconstruction.

1908, had not been removed at the time the picture was taken. The I-beams and 1¾-in. cables were in place and can be readily seen. The completed pier appears to be practically a duplicate of Pier 3, shown on Fig. 18.

The reinforced concrete shell placed around Pier 4 is practically a duplicate of that placed at Pier 3. It is shown on Plate VII. The tie-bars at the bottom were passed through the pockets in the crib and concreted in, instead of being passed through the pier. At this pier the ends of the spans were shifted sideways to permit the construction of each concrete pedestal in one piece.

As Pier 5 had moved north somewhat in the past, as a result of its own weakness and the thrust from Pier 4, anchors were buried in Pier 4 and the cables shifted ahead, Pier 4 now acting as an anchor for Pier 5.

Reinforcement of Top of Pier 5 in 1913.

The small tops of these piers, their repeated movement, and the frequent shifting of the spans, all contributed to a very poor condition of the bridge seat on Pier 5, on which the stones were breaking out under the end of the south span which rested close to the edge of the pier.

To strengthen the bridge seat, the top 10 ft. of the pier was encased in a reinforced concrete jacket about 9-in. thick, as shown by Fig. 32. The sides were connected by passing the reinforcing rods around the curved ends of the pier and also by three sets of rails set vertically in the reinforced concrete at the middle and quarter points of the length of the pier. The lower ends of these rails were attached to the pier by 2-in. anchor-bolts set in grouted holes. Their upper ends were connected by 2-in. rods extending across the pier tops in channels cut in the bridge seat and filled with concrete. All cracks in the pier top were carefully grouted. The bridge seat appears to have taken on a new lease of life.

It is expected that, as Pier 5 is quite a distance from the ordinary channel, there is little likelihood of the material surrounding it washing away, and no trouble is apprehended unless that should occur.

Following the reconstruction of Piers 3 and 4 and the reinforcement of Pier 5, the entire bridge, which was badly out of line and level, was lined and leveled. It was found that the draw-span was

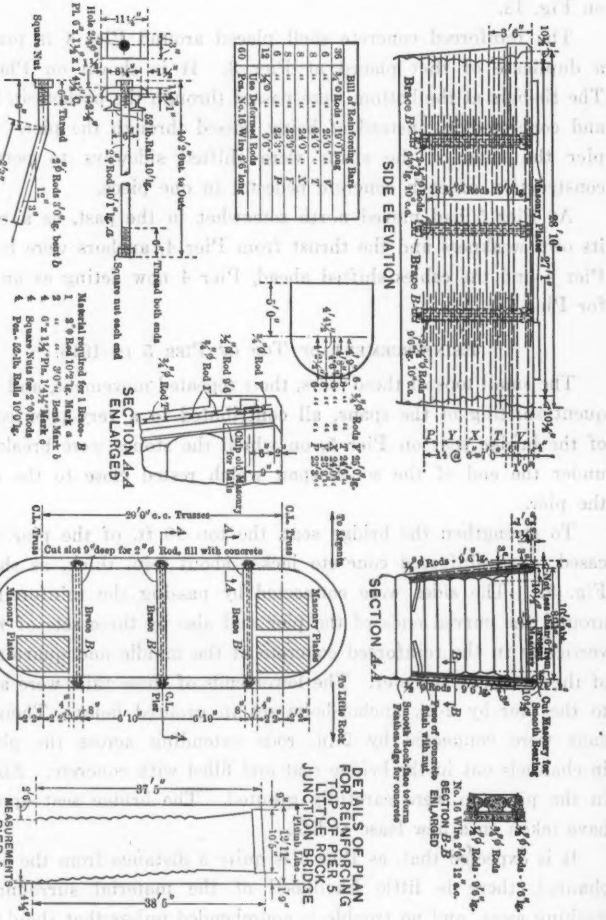
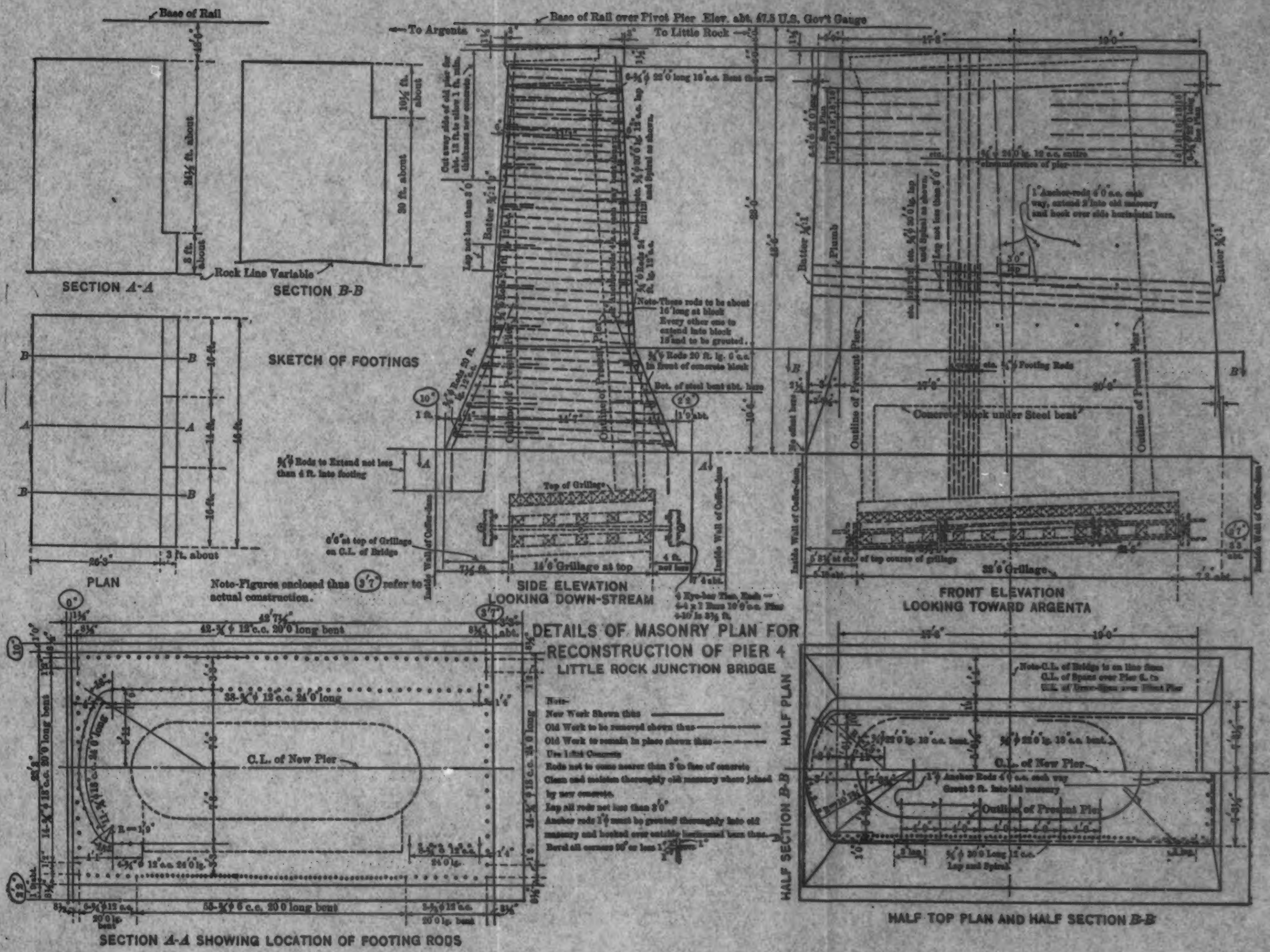

SIDE ELEVATION
SECTION A-A
SECTION A-A ENLARGED
PLAN
3 BRACES REQ'D MARK B
DETAILS OF PLAN FOR REINFORCING TOP OF PIER 5 JUNCTION BRIDGE LITTLE ROCK
MEASUREMENTS PIER 5
June 18, 1912.

FIG. 32.

PLATE VII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXIX, No. 1325.
SMITH ON
REPAIRING BRIDGE PIERS.



DETAILS OF MASONRY PLAN FOR
RECONSTRUCTION OF PIER 4
LITTLE ROCK JUNCTION BRIDGE

somewhat twisted and one end hung lower than the other. As far as could be done without injury to the span, the inequalities were corrected.

The work on the three piers and spans cost about $100 000, which is much more than it would have cost had the work on Pier 3 been started earlier and the floods avoided.

The reinforcement of Piers 3 and 4 was executed on a force-account basis by Bates and Rogers Construction Company. The adjustment of the spans and the reinforcement of Pier 5 were executed by the Railway Company forces.

Instrumental Observations.

Various systems of observations had been used to get a record of the extent and character of movement, but they had not been closely followed up, so that the old records of movement are intermittent. No figures are available previous to 1905, and measurements prior to 1908 were not strictly comparative because the errors of observation were often greater than the movement, but over any extended period they indicated the general trend of the movement. In March, 1908, triangulation monuments were placed on Pier 4 and on the river banks for monthly observations. The movement of the piers and spans, however, was such that it was necessary to change the points continually in order to get a clear view through the bridge, and as that rendered the observations of questionable value, this method was abandoned in 1910 for that in use at the present time. The character of the observations taken since that time is best described by the following copy of the instructions issued to observers on July 15th, 1910:

"Instructions for Taking Observations on Piers of Little Rock Junction Bridge, Little Rock, Arkansas.

"St. Louis, Mo., July 15th, 1910.

"Character of Observations.

"The observations consist of four independent determinations, as follows:

"*Line.*—Determining the location of two points on each pier by line relative to fixed monuments on shore.

"*Distance.*—Determining the location of said points by measurements from fixed monuments on shore.

"*Grade.*—Determining the elevation of said points with reference to established bench-marks.

"*Soundings.*—Determining profile of river bed entirely across the river at the center line of track and at the up-stream and down-stream ends of the piers.

"General Remarks.

"The observer must provide himself with a complete copy of these instructions and accompanying drawings, and study them carefully before making the observations.

"The two points on each of the piers consist of copper plugs, ¾ in. in diameter, set in the masonry, located and placed as shown on Sheets 1 and 4. The fixed monuments on shore consist of short sections of old rails set in the ground, located and placed as shown on Sheets 1, 2, 3, and 4.

"The points on the piers and the monuments on the Argenta side of the river being several feet below the level of the track, it is necessary to use a plumb-bob over each point. In order to protect the plumb-bob and string from atmospheric disturbances, a box, open at both ends and cut to proper length, as shown on Sheets 4 and 5, has been provided for each point. The boxes for Monuments *I* and *K* are fixed in position at the top end by hinges screwed to the under side of the bridge. This enables the boxes to be swung aside when desirable.

"The boxes for the points on the piers are marked *x*, *y*, etc., to correspond with the point over which they should be used.

"The boxes are held in place by cleats connecting their tops to the ends of the bridge ties. Those on the west or up-stream end of the piers extend 6 in. above the top of the floor-beams, those on the east or down-stream ends of the piers extend up to the under side of the plank walk. The boxes are provided with adjustable cross-arms for supporting the plumb line. The cross-arms for the west boxes may be tacked directly to their top; those for the east boxes may be tacked to blocking supported on the plank walk.

"Points are located on these adjustable cross-arms directly over the points on the piers by means of a plumb-bob suspended through the box directly over the respective points on the piers.

"Monuments *B*, *C*, *I*, and *K* are to be used for establishing the reference lines.

"The direction and character of the wind, the nature of the weather, distance base of rail to water surface, temperature, and any other conditions affecting the observations must be recorded on each occasion.

"Method of Procedure for Line.

"Set the transit over Monument *C* and check line *A B C D*; then set it over *H* and check lines *G H I* and *H J K L*; set over *M*

and check lines *M J I* and *N M K*. If these lines check, the monuments may be assumed to be correct. With transit at *M* and foresight on *I* the location of x_7 may be checked.

"Next move transit up to deck of bridge and set it over point *K*, sighting on *C* and establishing line at points, y_7, y_6, y_5, and y_4; then over point *I*, sighting on *B* and establishing points, x_6, x_5, x_4, and x_3.

"Next move transit to other end of bridge and set it over *C*, sighting on *K* and establishing points, y_1, y_2, y_3, and y_4.

"The distances between the points thus established and the original points formerly established on the copper plugs measured on the cross-arms, at right angles to the line of sight, must be recorded, as these distances indicate the movement of the piers in an east and west direction, at right angles to the direction of the bridge.

"This observation must be taken at least twice, the observers changing places and repeating all operations until their observations check.

"Method of Procedure for Distances.

"The measurements from fixed point on shore are made along lines *B I* and *C X*, points *B* and *C* being taken at Station 0, and the station of each of the points, *x*, *y*, etc., as well as *I* and *K* being recorded.

"On line *C K* the tape is permitted to rest on the tops of the floor-beams; on line *B I* (which follows the footwalk) wood blocks about 4 in. high must be placed over each floor-beam to reproduce the conditions of support obtained on line *C K* and to eliminate the effect of the temperature of the walk on the tape.

"The distances are measured with a 200-ft. tape stretched to a tension indicated as 15 lb. on a spring balance. Monuments S_1 and S_2 have been carefully set 200 ft. apart for standardizing the tape, which is correct at 62° Fahr. For other temperatures the measured distances must be increased (+) or decreased (—) by the amounts shown in the following table.

"Average Temperature.

Meas.	12	22	32	45	52	62	72	82	92	102	112
Bx	— .057	— .045	— .034	— .023	— .011	0	.011	.023	.034	.045	.057
Bx	— .112	— .089	— .067	— .044	— .022	0	.022	.044	.067	.089	.112
Bx	— .174	— .139	— .104	— .069	— .035	0	.035	.069	.104	.139	.174
Bx	— .258	— .206	— .155	— .103	— .052	0	.052	.103	.155	.206	.258
Bx	— .340	— .271	— .203	— .135	— .068	0	.068	.135	.203	.271	.340
Bx	— .422	— .338	— .253	— .168	— .084	0	.084	.168	.253	.338	.422
Bx	— .466	— .373	— .280	— .186	— .093	0	.093	.186	.280	.373	.466
Cy	— .053	— .043	— .032	— .021	— .011	0	.011	.021	.032	.043	.053
Cy	— .109	— .087	— .065	— .043	— .022	0	.022	.043	.065	.087	.109
Cy	— .170	— .136	— .092	— .068	— .034	0	.034	.068	.092	.136	.170
Cy	— .254	— .203	— .152	— .101	— .051	0	.051	.101	.152	.203	.254
Cy	— .338	— .270	— .203	— .135	— .068	0	.068	.135	.203	.270	.338
Cy	— .418	— .335	— .251	— .167	— .083	0	.083	.167	.251	.335	.418
Cy	— .461	— .368	— .276	— .184	— .092	0	.092	.184	.276	.368	.461

"Method of Procedure for Grades.

"The top of the steel rail in Monument C is the bench-mark elevation. For purposes of checking, any other monument may be used, the elevations being shown on Sheets 2 and 3.

"Set the level about half way between C and the south abutment, use point y for turning point. Move level to middle of first span and use point y for turning point. Continue across all the spans and back in this manner, checking on bench-mark C and taking readings on all points on piers in each direction for a check.

"In case the levels check upon returning to C and the two readings on each point check, this observation need not be repeated. Otherwise it must be repeated until a check is obtained.

"Method of Procedure for Soundings.

"Soundings shall be taken along the lines $P\ P_1$; $P_2\ P_3$, and $P_4\ P_5$, shown on Sheet 6, preferably by use of a lead line from a boat, except where the ground line is above water, where it can be taken by a weighted tape from the bridge floor. The distance from base of rail to water surface shall always be recorded.

"Soundings shall be taken close to the foundation under each pier, but far enough out to miss it, and at every second floor-beam of each span (about 50 ft. apart).

"This observation for soundings need be taken only once, there being no necessity for its repetition."

Fig. 33 shows the location of the base lines, points on pier, transit points, and other monuments. No movement has been detected in the pivot pier since the repairs in 1898; no movement has ever been detected in Piers 1, 6, and 7.

Fig. 34 shows the movement curve for Pier 3; practically no movement was detected up to 1908. In that year, following the application of the rods and yokes to the ends of the span resting on Piers 3 and 4, Pier 3 moved north (parallel to the bridge) about 10 in. from 1908 to 1913, during which period the reconstruction was carried out, since which work the pier has been stationary. No movement east or west (at right angles to the bridge) was detected in Pier 3 until the summer of 1910, when, for no apparent reason other than the deep scour of the river around the pier, it started up stream at the rather rapid rate of 9 in. in 14 months, the up-stream end settling several inches. This movement brought about the reconstruction of the pier in 1911 and 1912, during which it was found possible, by controlling the underpinning, to make the pier partly right itself, the curve showing that the top moved back (east) about 6 in. during the work.

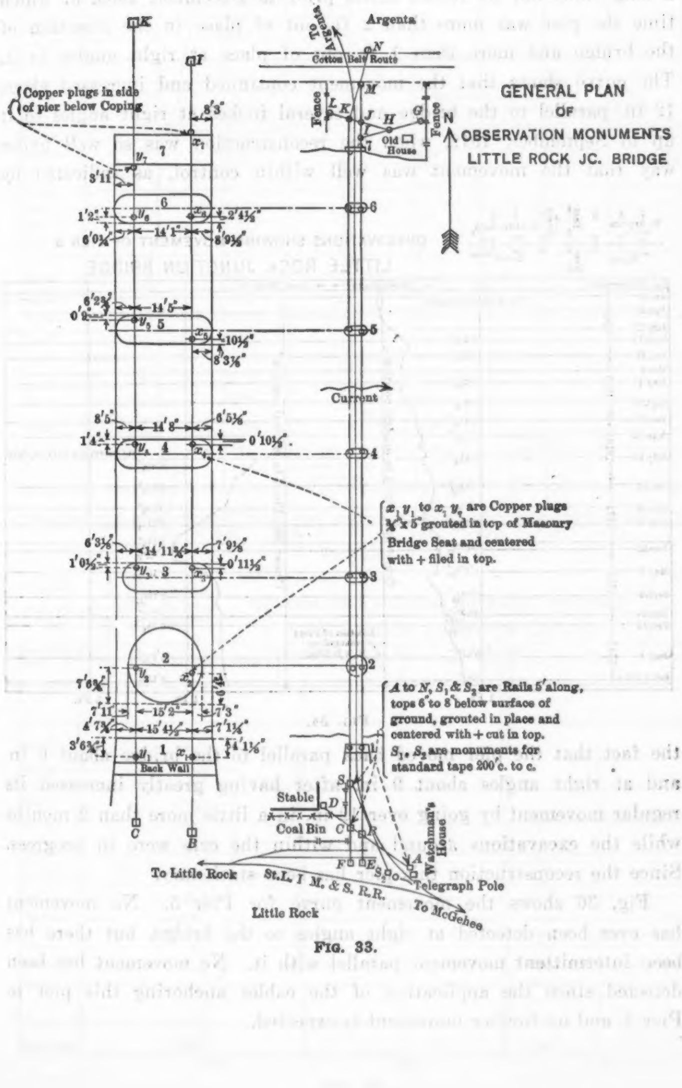

GENERAL PLAN
OF
OBSERVATION MONUMENTS
LITTLE ROCK JC. BRIDGE
Argenta
To Argenta
Cotton Belt Route
Fence
Old House
Copper plugs in side of pier below Coping
Current
Back Wall
x_1 y_1 to x_7 y_7 are Copper plugs ¾" x 5" grouted in top of Masonry Bridge Seat and centered with + filed in top.
A to N, S_1 & S_2 are Rails 5' along, tops 6" to 8" below surface of ground, grouted in place and centered with + cut in top.
S_1, S_2 are monuments for standard tape 200' c. to c.
Stable
Coal Bin
Watchman's House
Telegraph Pole
To Little Rock
St.L. I. M. & S. R.R.
Little Rock
To McGehee

FIG. 33.

Fig. 35 shows the movement curve for Pier 4. The position of the pier in 1898 made it evident that the movement had taken place for a long time, but no record exists prior to December, 1905, at which time the pier was more than 2 ft. out of place in the direction of the bridge and more than 1 ft. out of place at right angles to it. The curve shows that the movement continued and increased about 12 in. parallel to the bridge and several inches at right angles to it up to September, 1912, when the reconstruction was so well under way that the movement was well within control, as indicated by

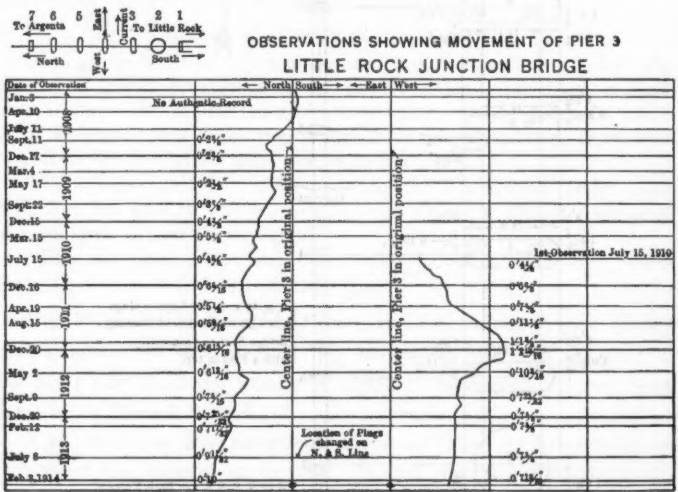


Fig. 34.

the fact that the pier moved back parallel to the bridge about 6 in. and at right angles about 9 in. after having greatly increased its regular movement by going over 12 in. in a little more than 2 months while the excavations around and within the crib were in progress. Since the reconstruction this pier has been stationary.

Fig. 36 shows the movement curve for Pier 5. No movement has ever been detected at right angles to the bridge, but there has been intermittent movement parallel with it. No movement has been detected since the application of the cables anchoring this pier to Pier 4, and no further movement is expected.

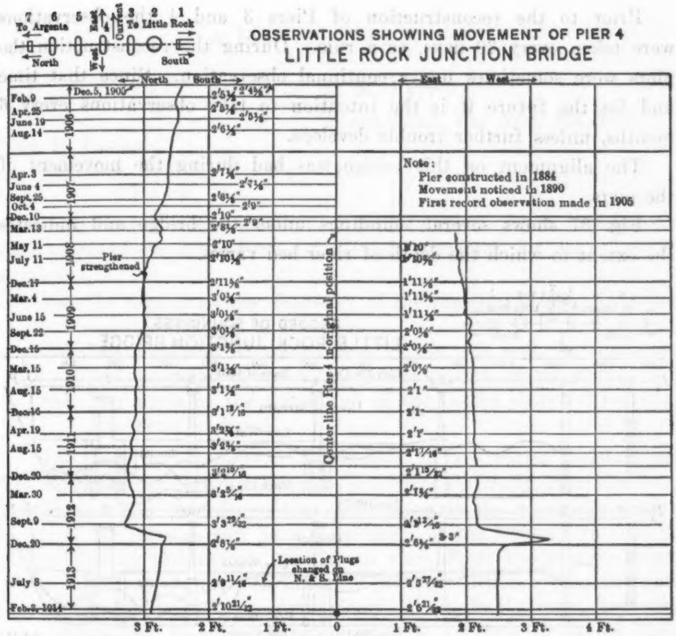

7 6 5 4 3 2 1
To Argenta
To Little Rock
North
South
East
West
Current
OBSERVATIONS SHOWING MOVEMENT OF PIER 4
LITTLE ROCK JUNCTION BRIDGE
North
South
East
West
Dec. 5, 1905
1906
1907
1908
1909
1910
1911
1912
1913
Feb. 9
Apr. 25
June 19
Aug. 14
Apr. 3
June 4
Sept. 25
Oct. 4
Dec. 10
Mar. 13
May 11
July 11
Dec. 17
Mar. 4
June 15
Sept. 22
Dec. 15
Mar. 15
Aug. 15
Dec. 16
Apr. 19
Aug. 15
Dec. 20
Mar. 30
Sept. 9
Dec. 20
July 8
Feb. 3, 1914
Pier strengthened
Note:
Pier constructed in 1884
Movement noticed in 1890
First record observation made in 1905
Center line Pier 4 in original position
Location of Plugs changed on N. & S. Line
3 Ft.
2 Ft.
1 Ft.
0
1 Ft.
2 Ft.
3 Ft.
4 Ft.

FIG. 35.

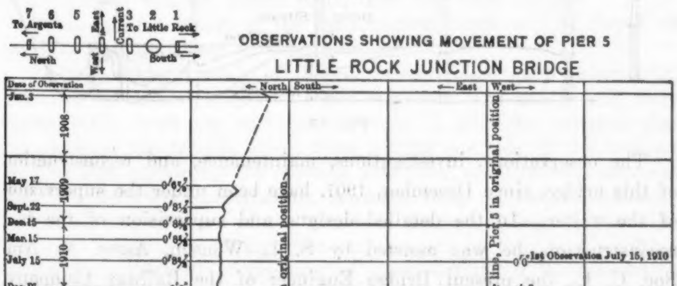

7 6 5 4 3 2 1
To Argenta
To Little Rock
North
South
East
West
Current
OBSERVATIONS SHOWING MOVEMENT OF PIER 5
LITTLE ROCK JUNCTION BRIDGE
Date of Observation
North
South
East
West
1908
1909
1910
1911
1912
1913
Jan. 3
May 17
Sept. 22
Dec. 15
Mar. 15
July 15
Dec. 16
Apr. 19
Sept. 13
Dec. 20
June 4
Sept. 9-12
Dec. 20
July 8
Feb. 3, 1914
Center line, Pier 5 in original position
1st Observation July 15, 1910
1 Ft.
1 Ft.
1 Ft.

FIG. 36.

Prior to the reconstruction of Piers 3 and 4 the observations were taken every 30 days, as a rule. During the reconstruction the piers were sometimes under continual observation. Since that time and for the future it is the intention to take observations every 6 months, unless further trouble develops.

The alignment on this bridge was bad during the movement of the piers.

Fig. 37 shows several soundings under the bridge and indicates the extent to which the depth of river bed varies.

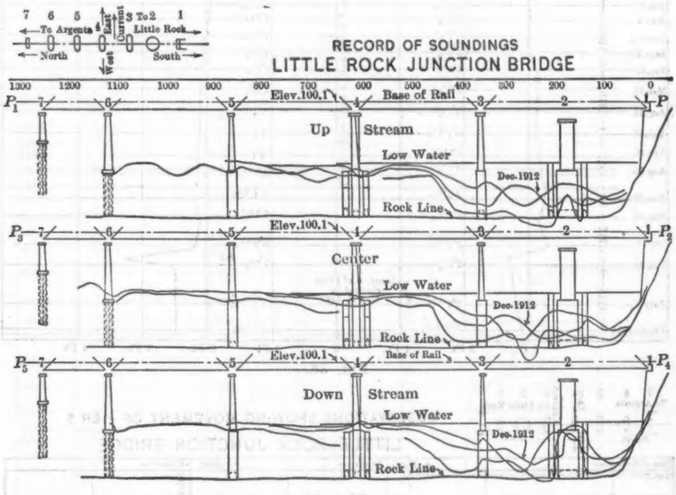


FIG. 37.

The observations, investigations, maintenance, and reconstruction of this bridge, since December, 1907, have been under the supervision of the writer. In the detailed designs and supervision of the last reconstruction, he was assisted by S. L. Wonson, Assoc. M. Am. Soc. C. E., the present Bridge Engineer of the Railway Company, and in the execution of the work on the ground all credit is due to Mr. F. L. Marshall, Assistant Engineer. Their courage, energy, persistence, and ingenuity made a successful accomplishment out of what might have been a deplorable failure.

APPENDIX A

REPORT OF W. M. PATTON.

"BLACKSBURG, VA., April 9th, 1898.

"MR. E. FISHER,
"Engineer, B. & B., Mo. Pac. Ry.,
"Pacific, Mo.

"DEAR SIR: The examination of the condition of the foundations of the piers of the Little Rock Jct. Bridge were made for the following purposes:

"1st. To determine the character of the material surrounding the caisson and crib of the piers.

"2d. The condition of the timbers in the crib and the character and condition of the filling in the pockets of the crib.

"3d. The depth of the water and of the earthy material between low water and rock.

"The first two have been determined to my entire satisfaction. The third has not been determined completely and fully, this, however, is only necessary in order to give definiteness to the maximum depths to which the caissons will have to be sunk, as the basis of an unconditional bid by Contractors, and in no wise affects the proper design of the structure required in order to prevent further settlement and to enable us to level the present piers. These depths have, however, been determined to such an extent that I feel safe in saying that the maximum will not exceed 40 ft. from low water surface.

"The detailed report of the examinations made will be found on separate sheets enclosed in this.

"After a careful study of the conditions, based upon information obtained, the examinations of other engineers, and finally and more satisfactorily from my own examinations, I am fully satisfied that the cause of the trouble arises from resting one side of the caissons on rock and supporting the opposite side upon some inferior material and this defective construction, rendered more so by the necessity of placing the masonry of the structure nearer one edge of the crib, owing to locating or sinking the caissons out of line, and resulting in a greater intensity of pressure on some portions of the foundation bed than upon other portions.

"The examinations indicate that the masonry is only a few inches (in fact practically on one side of the crib) from one side of the crib, whereas it is only 2 ft. 8 in. from the edge of the opposite side, consequently the axis of the pier is fully 1 ft. 4 in. nearer the west and north sides of the caissons than the east and south sides. The bottom area of the caisson is 1 024 sq. ft.; the estimated load (pivot pier) is 4 496 500 lb., equivalent to a load of 4 400 lb. per sq. ft. This is

nearly, if not quite, the maximum allowable unit pressure. The eccentricity of the load gives as the maximum pressure over 4 300 lb. pressure per sq. ft., which is in excess of the allowable load upon anything but solid rock. This condition is more noticeable at Pier 4 than at Pivot Pier. At Pier 4 the uniform pressure per square foot of foundation bed is 6 200 lb., and owing to the eccentric loading the maximum pressure per square foot must be over 7 000 lb., a unit load far beyond anything permissible.

"These conditions fully account for the settling of the piers, here it may be asked, why has the settlement been toward the west and not toward the north, if at both of these points the rock is at a greater depth than at the south and east corners? It may be answered that:

"1st. The filling between the bottom of the caisson and the rock at the north corner may be better than at the west corner.

"2d. The west corner, being the up-stream corner, is scoured out to a greater depth than the north corner, thereby reducing to that extent the supporting power of the material.

"The depth of water at the east corner was found to be 16 ft. 9 in., at west corner 22 ft. 5 in. and this, notwithstanding the large amount of rip-rap deposited at and on either side of west corner, and, moreover, other conditions might exist as to why this settlement to the west occurred, rather than toward the north, even assuming that the elevation of the rock at the west corner is the same or even higher than the north corner. We are, however, dealing with fact not a theory, and whatever may be the difference of opinion as to the cause of the trouble, the important question is, what is the proper and best remedy, under known, not probable conditions. Clearly any design with this object in view should enable us to solidify the present material under the westerly half of the caisson, or remove that material and substitute some more solid material for it:

"1. To solidify the material at present between the bottom of the caisson and the rock. The only practicable and available method is known as the Harris' process, which consists in injecting pure (liquid) cement under pressure into the material, thereby converting it into concrete in place. This method while perhaps never employed in a case exactly similar to the one under consideration, has proved to be effective under other conditions and where the original material is clean sand, I am satisfied it is a safe and effective method. The examination showed a remarkably clean sand and gravel from the bed of the river to rock; at the east corner a thin layer of clay and silt, at about 7 ft. above the rock; but as it would only be necessary to apply, at any rate for the present, the process to that half of the caisson from the south through the west to the north, the presence of this clay at the east corner would be of no consequence, and its

distance from the rock would render it not worthy of consideration at all. The conditions are, therefore, exceedingly favorable for the application of the Harris' process. It will be a, comparatively speaking, rather inexpensive process, even recognizing it as patented, probably \$10 000 would cover the entire cost. I do not, however, recommend this method, owing to the uncertainty that would necessarily exist as to the results obtained, which could only be determined by time, and in addition it would preclude the possibility of raising the pier to its proper vertical position. If you see proper you can correspond upon the subject with the Harris Company of New York.

"2. To remove the present material under the caisson and substitute it with concrete; the available methods to accomplish this are:

"1st. The Poetsch-Sooysmith freezing process.

"2d. The ordinary coffer-dam.

"3d. The pneumatic caisson, using the old caisson and crib for one of its walls.

"4th. An independent caisson and coffer-dam constructed entirely or partly around the old caisson and crib.

"1st. An ordinary coffer-dam let a few feet into the bed of the river and freezing a wall of the material below to the rock. This method can be relied upon as effective, and the only consideration is in respect to the cost. You can correspond with Wm. Sooy-Smith of Chicago or Charles Sooysmith of New York.

"2d. The ordinary coffer-dam is probably hardly worth considering, as it would be extremely difficult to make a water-tight dam in the kind of material around these piers and of the depth required. There would be many risks to incur of failure and the saving in cost, if any, would hardly justify the employment of such an uncertain method.

"3d. The method that at first seemed to me the best, surest, and most economical was to use a pneumatic caisson, making the old crib and caisson one of its walls. My own examination of the crib and its filling, which entirely corresponds with the report of the divers, discloses the fact that the crib is filled with small rip-rap, sand, and gravel, the latter in all probability was washed in subsequently to the completion of the structure, through the open sides and pockets. This condition convinced me that at least it would be a very risky method, and the risk is so great that I do not consider it justified by the saving in cost, if it should prove successful.

"4th. I have, therefore, been compelled to recommend the adoption of an entirely independent caisson, entirely or partly surrounding the old crib and caisson, leaving a working space of 6 ft. between the old and new caisson. The working designs for this structure with bill of material, estimate of cost and full detail, are sent by express to your address.

"While this design in many of its main features is similar to that designed for many piers constructed by me, there are some novel features introduced which are justified and demanded by the novel conditions, to which it is adapted. The design commends itself for the following reasons:

"1st. It gives absolute assurance of success, in exposing the material between the bottom of the caisson and the rock, and affording the opportunity of removing that material and substituting in its place concrete, which will unquestionably prevent further settling, due to the yielding material at present under the caisson.

"2d. It affords the opportunity of raising and leveling the pier in its original and proper position.

"3d. It affords the opportunity of making a thorough examination of the timbers of the crib and the caisson, and of repairing any damage done to the crib, arising from the settling.

"The report of the diver, confirmed by my own examinations, leads me to believe that some damage has been done to the upper timbers and the sheathing planks during the settlement, and I consider it important to either restore the sheathing plank or to fill the pockets to a certain distance inward on the several faces to hold the filling in the crib. This being done, the present filling, although of small stone, sand and gravel, will no doubt be sufficient and satisfactory.

"During the examinations recently made, a number of 12 by 12-in. timbers were bored entirely through at depths varying from 10 to 30 ft. below the water surface, and in every instance the timber was found to be firm and sound.

"The design shows the new caisson as constructed entirely around the old one. It may not be necessary to construct it around more than one-half of the old one; that is, around the west corner and extending the length of the side toward the north and south corners, respectively. This, however, forces us to utilize the old caisson as one wall of the coffer-dam, and it is possible, if not probable, that this will not hold the water back, so that the space between the two caissons may be pumped out. However, this half can be sunk as a section and the other half sunk, if found necessary; no change in the design will be necessary. It will be necessary to make a water-tight connection between the ends of the new caisson and the old caisson at the north and south corners. It will also be necessary to bolt two channel irons at each end of the new caisson, so that, in the event of adding the second section around the east corner and extending the south and north corners, the two sections can be united so as to form a water-tight joint.

"The estimated cost of the caisson and coffer-dam constructed entirely around the old caisson is about $40 000, and of the half caisson and dam about $20 000 to $25 000.

"The filling above the deck of the caisson required to sink the caisson may be of any material desirable, as far as practicable, the material taken out of the working chamber during the sinking may be used for this purpose, and in fact this should be required, or at any rate no payment should be made for the filling if the contractor prefers to get it from other sources; also the material between the new caisson and the old one can be dredged during the process of sinking and used for filling. This, however, should be done with judgment, as it will evidently be unwise to remove the material at and near the west corner of the old crib until we have at least given some temporary support to the caisson to the west of the center line of the pier, or that diagonal line joining the north and south corners.

"Owing to the relatively small depth of the earthy material, through which the sinking is to be done, it is possible that no filling above the deck of the caisson will be necessary. Filling this space with water may answer every purpose. It may not be necessary to fill the working chamber itself if a water-tight joint can otherwise be formed between the bottom of the caisson and the rock. If not, a concrete or clay puddle must be used of sufficient thickness to secure water-tightness. Owing to the shaly and inclined nature of the rock, it may be necessary to cut a longitudinal trench in working chamber and to fill this with puddle or concrete in order to cut off seeping water along the seams of the rock. These are matters to be determined by observation and the exercise of good judgment at the time. It is proposed to obtain water-tightness of the structure above the deck of the caisson by means of caulking the timbers and plank walls, and not by means of the filling material. One or both walls should be caulked for this purpose.

"The air-tightness of the working chamber must be secured by thorough and perfect caulking. These and other similar matters will be embodied in the outline of the specifications accompanying the drawing. You will note the upper half of the structure is so designed and connected that it can readily be removed. The lower half being below the bed of the river, or at least of sufficient depth below the water surface not to interfere with navigation, can be left in place. I, therefore, respectfully submit the accompanying drawings, marked Sheets 1, 2, and 3, accompanied with the following suggestions and recommendations.

"1st. Sinking the caisson in two sections, the one section extending from the south corner around through the west to the north corner, which may be all that will be required, and sinking the second section from the south corner around through the east to the north corner, if found necessary, owing to the open condition of the old crib.

"2d. Excavating the material from the north and south corners of the old crib and between the old and new structures to a point, say,

half way between the south and north, respectively, and the west corner, removing the loose material from beneath the old structure and replacing with concrete, or some temporary support.

"3d. Then excavating the material from, at and near the west corner, exposing the space below the old structure, removing the loose material and replacing the concrete or some temporary support. This temporary support contemplates raising the pier to its original and vertical position.

"4th. If practicable, I would suggest supporting the superstructure on temporary supports during the time occupied in cleaning out the material between the old and new structures and until the permanent filling is in place and has had a reasonable time for hardening.

"5th. For the purpose of lifting the pier, I would recommend the employment of hydraulic jacks. It is estimated that hydraulic jacks having a main cylinder of 3.5 in. diameter and a pump plunger of $\frac{3}{4}$ in. diameter, will raise 20 tons of 2 240 lb., without excessive steam pressure. Upon this basis there will be required from 30 to 35 jacks, assuming that the superstructure is temporarily lifted from the pier; the cost of this portion of the work will be little more than the cost of the jacks.

"6th. With the structure supported on jacks the removal of the loose material and substitution of concrete (for which the best Portland cement should be used) can be proceeded with without risk or danger; otherwise this work should be carried on in sections, and precautions taken not to reduce the supporting resistance, as it now exists, at and near the west corner, until the caisson is well supported, two-thirds to three-fourths of its area. The amount of concrete estimated for filling under the caisson will not exceed from 150 to 200 cu. yd.

"7th. As the design submitted will afford ample opportunity for a full examination of the caisson and crib, I would suggest filling the open spaces on the sides with concrete to the depth of from 12 to 18 in., forming a wall to hold in place the present loose and imperfect filling. A good sheeting of timber, well bolted to the timbers of the crib, will probably answer every purpose.

"The character and extent of this work necessary, can only be determined at the time.

"With the drawings you will find a bill of material, estimate of concrete required, and a detail report of the character and extent of the examinations and the information obtained.

"Trusting that you will find everything in satisfactory shape, I am,

"Yours truly,

"W. M. Patton.

"P. O. Box 209

"Blacksburg, Va.

"In the event that it is deemed better or found necessary to entirely enclose the old structure with caisson and coffer-dam, it may be found more economical and satisfactory to excavate the rock at and near the south and east corners, and to lower this portion of the structure, thereby bringing the pier to its original vertical position, rather than lifting the westerly portion by means of hydraulic jacks."

REPORT IN DETAIL OF THE EXAMINATIONS MADE AT PIVOT PIER, LITTLE ROCK JUNCTION BRIDGE, MO. PAC. RAILWAY.

"*South Corner.*—Depth of water, 28 ft. 11 in. below low-water surface. Depth of rock, 35 ft. 6 in. below low-water surface. Drilled into rock 4½ in.

"The pipe was driven through fine sand, followed by a layer of coarse white sand mixed with black scales, feeling and looking like graphite; immediately overlying the rock was found a layer of fine and coarse gravel mixed with coarse sand. The rock is commonly called slate, is a black shale, which yields readily to the drill. Time occupied in setting and sinking pipe about 3½ hours.

"*East Corner.*—Depth of water, 13 ft. 6 in. below low-water surface. Depth of rock, 37 ft. 6 in. below low-water surface. Drilled into rock 3 in.

"The pipe was easily driven in medium sand, passing into sand mixed with gravel to a depth of 23 ft. below low water. This continued to a depth of 30 ft., where sand and gravel mixed with a soft but tenacious clay was found. At 34 ft. down a rather fine sand was found, mixed with clay. Rock was reached at 37 ft. 6 in. below low-water surface. There was found no well-defined stratum of gravel at this corner. The rock was of the general character, as already indicated. Time occupied in sinking about 4 hours.

"*North Corner.*—Depth of water, 22 ft. 7 in. below low-water surface. Depth of rock, 39 ft. below low-water surface. Drilled into rock 3½ in.

"Pipe was easily driven through medium sand to a depth of 29 ft. 3 in. below low water. At 26 ft. 9 in. was found fine and coarse sand mixed with fine and coarse gravel, and black scales, which continued for about 2 ft. to a depth of 28 ft. 9 in., the driving being somewhat difficult to a depth of 30 ft. 9 in., below this the pipe entered a fine flowing sand, which rapidly filled up the pipe for several feet whenever the jet was stopped. This material consisted of fine white sand, suggesting quicksand by its readiness to flow. From this point it was necessary to run hammer and jet simultaneously. The large pipe followed the small pipe readily, but would fill up if jet stopped. This fine sand continued to a depth of 39 ft. Drilled into the rock about 3 ft. Rock similar to that at other places. Time about as at other points.

"*West Corner.*—At and near this corner large quantities of rip-rap have been deposited. It was impracticable to find an opening through this sufficiently large to permit the passage of the 3-in. pipe. The small jet pipe was easily run to a depth of 33 ft. 6 in. below low-water surface. Further progress was prevented by a bed of gravel and the friction on the pipe above. Several attempts were made to force the 3-in. pipe through the rip-rap, which invariably resulted in the squeezing and mashing of the large pipe and the parting of the small pipe. On Saturday, March 12th, however, we succeeded in getting the 3-in. pipe through the rip-rap.

"Depth of water 19 ft. 3 in. below low-water surface.

"At 24 ft. down, the pipe brought up on a log. This log was bored through, it proving to be 12 in. thick. The timber was found to be firm and sound. The 3-in. pipe was driven through this log, and at 26 ft. 10 in. the pipe struck another log, which was partly bored through, when further work was stopped at 6 P. M. It had been raining hard all day, which continued through the night. The river rose rapidly, so much so that it was necessary to remove the barge to a place of safety. Work in the river was then abandoned for one week. This ended the first week's work. There were many delays, caused by time required to cut and thread pipes, and pump getting out of order, foaming of the boiler, etc. These were, no doubt, incident to and unavoidable in this kind of work, and it is not intended to attach any blame to any one, as all parties seemed disposed to do anything when called upon, and on the whole I was entirely satisfied with the work done the first week.

"Seeing no prospect of getting to work in the river, preparations were made to get into the crib within the well in the center of the pier, and on Thursday, March 17th, work was commenced. The well was found to be filled to about 11 ft. with silt, logs, and stone. After several efforts, a 3-in. pipe was forced to the timbers of the deck, and these timbers, about 2¼ in. thick, were bored through, admitting the small pipe without difficulty several feet below the deck and into the filling of the crib. This filling proved to be small stone, sand, and gravel. In attempting to drill through a stone the small pipe parted, resulting in loss of drill.

"On Monday, March 21st, the river having fallen a few feet, an effort was made to resume work in the river. A large portion of the day was consumed in getting the barge in position, and little was done subsequently, owing to the almost continuous foaming of the boiler, and consequent failure to supply requisite pressure. On Tuesday, with a rising river, a new pipe was sunk near the west corner, but it could not stand the constant vibrations caused by a strong current acting on some 40 ft. of unsupported length, and broke. At this

time the river had risen considerably, and it was deemed necessary to remove the barge to a place of safety.

"I desire to express my appreciation of the uniform kindness, consideration and faithfulness of officers and laborers on this work.

"With great respect,

"W. M. PATTON."

SPECIFICATIONS FOR THE CONSTRUCTION AND SINKING OF CAISSONS FOR THE PURPOSE OF REPAIRING THE PIVOT PIER AND PIER 4 OF THE LITTLE ROCK JUNCTION BRIDGE, MISSOURI PACIFIC RAILWAY.

"There will be constructed two caissons, the one for the pivot pier and the other for Pier 4. These caissons may be of such design and dimensions as will entirely surround the old piers of the bridge or will extend around any portion of these piers, as may be directed by the Engineer of B. & B. of the Mo. Pac. Ry. These caissons shall be constructed according to designs given on Sheets 1, 2, and 3 accompanying and forming a component part of these specifications, or according to such other design as may be accepted or approved by said Engineer of the Railway Company.

"Sheets 1, 2, and 3 give plan, elevation, section, and exterior and interior detail of the caisson and coffer-dam to be constructed entirely around the pivot pier.

"The structure is built of timber and iron; all timbers are 12 by 12 in. in cross-sections, excepting a few pieces 9 by 12 in. and 6 by 12 in., used for supporting braces, and 3-in. planks necessary for lining the caisson and coffer-dam. Exact lengths are given and where not given variable lengths are allowed. All necessary dimensions are given on the drawing and require no description. In the event that Engineer determines to construct a caisson of only one-half the dimensions given in the plan, no change in design will be required except closing up the free ends as indicated in the drawing.

"*Timber.*—All timbers used in the construction of the caisson and coffer-dam may be what is known as merchantable, and may be either pine or oak. *The outside verticals forming the lower half of the structure must be of the best long-leaf yellow pine and showing clear heart on the exposed face.* All timbers must be free from all defects that will impair their strength, such as rot, dotiness, or sponginess, large knots, or deep shake, or cracks of any kind. All framing and fitting must be done in a workmanlike manner and to the approval of the said Engineer.

"The upper and lower halves of the structures are to be built entirely independently and not bolted together in any manner. The two halves being held together solely by the 1¼-in. hook-rods, as indicated in the drawings. The lining plank in the caissons, as well as

in the coffer-dam, shall be planed on the edges for a caulking joint.

"*Caulking.*—The caulking of the inner plank of the air or working chamber shall be thoroughly done, so as to make the sides and top of the air chamber completely air-tight and paved over with tar or pitch. Oakum shall alone be used in caulking. The joints of the top deck course shall also be sufficiently caulked to form a water-tight surface. The verticals forming the sides of the caisson, or lower half of the structure, shall be caulked on the outside, so as to form a water-tight surface. The inner plank lining of the coffer-dam shall be caulked, so as to make water-tight surfaces, as reliance for keeping out water is placed entirely on this caulking.

"*General Construction.*—The exposed surfaces of the lower half of the structure are built of timbers 12 by 12 in. by 23 ft., thoroughly bolted to these on the inside are five layers of horizontal timber, one layer on top of the other, which are well bolted to each other; spiked to these horizontal timbers are two layers of 3-in. plank, the first course placed diagonally and the second or inner vertical and caulked. Resting on top of the horizontals and plank, is placed a solid layer of 12 by 12-in. timbers, bolted at the ends to the horizontals; on top of this another solid layer of 12 by 12-in. timbers, placed diagonally and bolted at intervals of about 5 ft. to the first course, with 1-in. square by 22-in. drift-bolts. Over this course another solid course of 12 by 12-in. timbers laid longitudinally and similarly bolted to the course below. These three courses constitute the deck of the caisson proper. To the underside of this deck a layer of 3-in. plank is spiked and caulked. Above the deck and between the vertical sides, an open crib of 5 courses of timber is constructed, bolted to the verticals, to the deck and to each other. Above the crib is one set of cross-braces between the verticals and resting on longitudinals, bolted to the verticals. Diagonal rods are used in spaces between these braces and in a horizontal plane. Another set of cross-braces are placed at the top of the outside verticals. This completes the lower half of the structure and may be denominated as the caisson.

"*Coffer-Dam.*—Sills are placed on top of the verticals on both walls of the caisson, and upon the sills, at a general interval of 5 ft., center to center, vertical posts 12 by 12 in. by 21 ft. are placed, and on top of these posts caps are placed. These posts may be spiked or bolted to caps and sills, or they may be mortised and tenoned. Two layers of 3-in. plank are then spiked to caps, sills, and posts on the inside. The first layer is placed diagonally and the second or inner layer horizontally and caulked.

"Cross-pieces are then placed over the caps and projecting 18 in. beyond outside verticals, through which the 1¼-in. rods with hooks, pass. These rods are connected with the I-bolts, let into the verticals. Plate washers are used at the top, as the two halves, upper and lower,

are to be pulled and held together by these hook-rods. The two joints between sills and top of the verticals of the caisson are to be well caulked after the two portions are pulled firmly together. An inner strap, countersunk, encircles the caisson, to which it is well spiked just above and resting against the I-bolts. There are three sets of braces between the walls of the coffer-dam, with diagonal rods between them and horizontal planes, as indicated in the plan and sections. These rods have swivels or couples at some intermediate position.

"The coffer-dam is framed in four sections, two of which are 70 ft. long and two are 44 ft. long. These sections are connected at the ends by diagonal rods with swivels and couplings, double posts being used at the junction of the sections; also 18-in. channels are let into, but not bolted to, these posts, in order to hold the sections together.

"Removing these channel irons and uncoupling the diagonal rods, sever all connections between the sections of the coffer-dam as well as the two walls of the same section, so that the upper half of the structure can be readily removed at the completion of the work. The lower half of the structure is intended to remain permanently in place. All other details, with bolts, straps, and braces, are shown on the drawings, and also the dimensions of the same, as well as the bill of material accompanying.

"*Iron.*—All iron is to be good, tough, ductile, and fibrous, except that all washers may be cast iron. The top washers for 1¼-in. hook-rods should be wrought-iron plate, about 1 by 6 by 6 in. In all cases the grip of screw bolts, that is, net length of washers, is billed. Proper allowance must be made for heads, nuts, and washers. The weights given are for full lengths, including heads, nuts, and washers. All bolts must be wrapped with oakum under head before driving, and at the threaded ends before placing nut and washer on.

"All drift-bolts are billed as 1 in. square. Round drifts 1 in. in diameter may be substituted, except for those bolts used in the outside verticals of the caisson.

"*Filling Material in Caisson and Coffer-Dams.*—All these caissons are only used for a temporary purpose and for convenience and certainty, and, after being sunk to the proper depth, serve only the purpose of coffer-dam, the material used in the filling is of secondary importance. The filling in the coffer-dam is only required to give weight necessary to sink the caisson and may therefore be any material. As far as practicable, the material taken out of the working chamber may be deposited above the deck of the caisson and in the coffer-dam, thus furnishing weight to sink the caisson. Also the material between the old caisson and new caisson may be excavated by dredging and likewise deposited in the coffer-dam, care being taken, however, not to remove the material at and near the west corner of the old caisson until it is safely secured by permanent supports, or

temporary supports, from further settling. It may not be necessary to entirely fill the coffer-dam in order to secure sufficient weight. If necessary or desirable, water may be let into the coffer-dam so as to supplement the weight of other material, in such case, the water can be ultimately pumped out, thus facilitating the removal of the coffer-dam.

"The filling in the working chamber of the caisson after it has reached the rock, or its final position, is only necessary to prevent leaking under the caisson. It must be such material, and in such quantity, and so placed as to prevent any under flow of water. No filling of any kind may be necessary, and will not be if the water surrounding the old caisson and between it and the new caisson can be pumped out and kept out. In this case some additional cross-bracing in the working chamber may be found necessary. It may be found necessary to fill the working chamber in the whole or in part with clay puddle, a water-tight joint between the caisson and rock being the important object.

"*Excavating the Material Between the Old and New Caissons.*—This may or shall be done in part during the sinking of the caisson, and the material thus excavated used to weight the caisson. In any event, the material at and near the west corner of the old caisson must not be disturbed or removed until a sufficient and safe support has been placed at other portions of the caisson. The material at and near the west corner can then be removed, thus exposing the entire surface of the old crib and caisson from its top to rock.

"*Raising or Leveling the Old Caisson.*—It is the intention of the Railway Company to lift and return the pier to its original and proper position, for which purpose the use of hydraulic jacks will be necessary. It will be necessary for this purpose, or at any rate may be advisable, to support the superstructure on trestles or some other form of temporary supports. The posts of the coffer-dam should not be used for this purpose. However, a sufficient number of additional posts may be inserted in the coffer-dam to carry the load. It will be better and safer, however, to support the superstructure by supports entirely independent of the coffer-dam.

"After inserting the jacks, with proper and safe bearing for uniform distribution of pressure, it may be found necessary to flood the coffer-dam, thereby reducing the weight to be lifted. In this case the pumps will have to be above the water surface and connected by means of small pipes to the main cylinders of the jacks. An additional number of jacks should be provided, and as the water is again pumped out, these should be brought into bearing so as to support the extra weight brought into action when the water is removed.

"In the event of constructing a caisson entirely around the old structure, it may be easier to excavate the rock from the south corner around through the east corner and as far as may be necessary toward

the north corner, and to level the structure by lowering the high portion of the caisson, rather than by raising the lower portion by means of hydraulic jacks.

"*Filling Under Old Caisson.*—In whatever manner the old caisson is restored to its proper position, or whether it is left in its present position, all loose material is to be removed from beneath the old caisson and good Portland cement concrete substituted. The concrete shall be composed of such material, mixed in such manner and placed and rammed in accordance with the approval and direction of the said Engineer of the Railway Company. Unless the structure is safely and securely supported by jacks, this work of underpinning must be done in sections, so as at no time to endanger the structure by further settling at any point. The jacks, if used, must be left in position until in the judgment of the Engineer the concrete has been sufficiently hardened to be able to carry the load with safety. The jacks can then be removed and the spaces filled with concrete, if the Engineer may so require.

"*Refilling Between Old and New Caisson.*—The space between the old and new caisson, from the rock to the bed of the river, will be refilled with material, if so required by the Engineer.

"*Removing the Coffer-Dam.*—The upper half or section of the structure, known as the coffer-dam, will be removed as already indicated.

"*General Remarks.*—In the bill of iron no mention is made of shafts, pipes, or other appliances required to sink the caisson, as these are regarded as belonging to the contractor's plant.

"The contractor shall furnish all material, tools, machinery, and apparatus of all kinds required to construct and sink the caisson, pump out water, and remove material. The contractor shall be responsible for any damage to the old structure, and shall take all risks incident to such work. The Railway Company shall be in no wise responsible for loss of life, damage to property, or interference with the safe navigation of the river. He will be responsible for all acts of his employees, and shall discharge any of his employees, when so directed by the Engineer, who fail or refuse to perform the work in accordance with the direction of the Engineer or his duly authorized representative.

"All material is to be approved and accepted by the Engineer before being used in the work. All work must be done in a thorough and workmanlike manner and to the approval of the Engineer or his duly authorized representative.

"*The Caisson for the Pivot Pier.*—The caisson for the pivot pier is composed of four sections; two of these are 70 by 13 ft. in plan and the other two are 44 by 13 ft., when the caisson entirely surrounds the old caisson. If it is determined to enclose only one-half of the old caisson, only three sections will be required: one 70 by 13 ft.;

and two 38 by 13 ft. All sections are 46 ft. 3 in. from bottom of caisson to top of coffer-dam, and 47 ft. 3 in. over all. Estimated cost on first plan $40 000, and on second $21 200.

"*The Caisson for Pier 4.*—The caisson for Pier 4 is composed of four sections; two of these are 70 by 13 ft. in plan and two are 27 by 13 ft., when the caisson entirely surrounds the old caisson. If it is determined to enclose only one-half of the old caisson, only three sections will be required, one 27 by 13 ft. and two 35 by 13 ft.; all sections are 46 ft. 3 in. from bottom of caissons to top of coffer-dam and 47 ft. 3 in. over all. Estimated cost on first plan $34 000, and on second $17 000.

"In either case the designs are essentially and substantially the same, only differing in axial length."

APPENDIX B

LETTERS OF A. J. TULLOCK ACCOMPANYING BID OF MISSOURI VALLEY BRIDGE AND IRON COMPANY FOR CARRYING OUT PATTON'S RECOMMENDATIONS AT PIVOT PIER AND PIER 4.

"LEAVENWORTH, KAN., Aug. 17, 1898.

"MR. E. FISHER,
"Engineer B. & B., Mo. Pac. Ry.,
"Pacific, Mo.

"DEAR SIR: Referring to your claims and specifications for the repairs of Piers 2 and 4 of the Little Rock Junction Railway Bridge, will say that I have gone into that subject quite fully and submit you herewith three separate propositions, covering the main portion of the work to be done in said repairs, leaving, however, certain parts of the work, the nature of which cannot now be determined, to be arranged at a later date when it can be clearly seen what is required. Such work can very well be done on a percentage basis, and probably so to best advantage.

"It is quite clear that the settling of the piers mentioned in the Little Rock Bridge arises either from the caissons not having been originally properly founded in solid rock, or from the crushing of the timbers in the cribs between the caissons and the masonry, as it is well known that these cribs were not well constructed when the work was built. Present appearances would indicate settlement due to the caissons not having been properly founded in the solid rock, rather than to the crushing of the cribs, but as against that, we have the written statements of the contractor who built the work, Mr. Barr, and of the Resident Engineer in charge of the work, Mr. Purdon, both reputable gentlemen, both of whom agree in saying that the caissons were properly founded in bed-rock and that the settlement of these two masonry piers must be due to crushing of the cribs. However, this can only be satisfactorily determined by actual examination, after sinking outside surrounding caissons as is now proposed.

"From a careful reading of your specifications, I conclude that, inasmuch as no concrete or filling of any kind is required in the caissons, that it was the intention of Prof. Patton that these caissons should be used simply for the purpose of getting down into the rock around the old piers to permit the underpinning or other repairs suggested by him, and that he contemplated relying entirely upon the underpinning or repairs made directly in the body of the old piers for permanent stability.

"It seems to me that we cannot safely rely on such underpinning for safety in this case, and it is very doubtful indeed if such underpinning can be done at all, without greatly endangering the entire

structure of the old piers. It is my opinion that the work to be done in this case should properly consist of sinking the caissons as proposed in your specifications, but that they should be filled with some material sufficient to provide permanent lateral stability, and that the space between the new caissons and the old piers should be excavated down to bed-rock, pumped out, and filled with concrete to a sufficient height to thoroughly support the old pier and prevent any lateral movement. This being done, and the outer caisson being sunk into rock so as to absolutely shut off any outward movement of soft material, which may underlie your present pier, the absolute stability of the pier against further settlement or movement of any kind will undoubtedly be assured, providing this settlement is due to lack of perfect foundation under the original caissons. I have therefore arranged my proposals with a view of carrying out this plan, as well as making them applicable to the programme apparently contemplated in your specifications, in case when the excavations are made, it might be found possible to make the repairs as suggested by Prof. Patton.

"I will not hesitate to say in advance, however, that I consider the method of repairing proposed entirely impracticable, and too dangerous to warrant its adoption, except under conditions so extremely favorable that we have no right to assume they will exist in this case.

"The caissons necessary to surround these old piers properly are necessarily so unusually large as to become very expensive as compared with the ordinary caissons used underneath piers of that kind in original construction, and in addition to the fact that a very large volume of material, both in construction and excavation, is involved, the peculiar form of these caissons is such that they must be very strongly constructed and braced in all directions to prevent danger of their becoming twisted in sinking, and being wrecked in that way. The cost of these caissons in place, therefore, becomes very much larger in proportion than the cost of ordinary caissons for new piers.

"We have figured out these caissons in detail, and submit you herewith blue prints of same, numbered respectively 3 928 and 3 929.

"I submit the following proposals for doing this work:

"*Case 1.*—This case includes furnishing, building, and sinking to the depth specified by you, the timber caissons and coffer-dams described in your specifications, and shown on our drawings, including the excavation of all material within the caissons to such specified depth, and the excavation and pumping out of the material between the new caissons and the old piers down to bed-rock, providing it can be excavated that far without endangering the old piers. This case, however, does not include the filling of any of the caissons with concrete.

"Furnishing, building, and sinking as above described,

"Pier 2 (draw pier) .... $28 600.
Furnishing, building, and sinking Pier 4... 25 600.

"*Case 2.*—Caissons and coffer-dam constructed and sunk precisely the same as in Case 1, and all necessary excavation same as in said case, adding thereto, however, the sealing of the caissons in the working chambers with Portland cement concrete to a height of 2 ft. above the cutting edge, all the way around, and remainder of the working chambers and caissons to be filled with pure sand:

"Pier 2, as above described..................$31 600.
Pier 4, as above described.................. 27 950.

"Steel caissons 6 ft. wide in the clear, height same as for wooden caissons, Cases 1 and 2, surmounted by wooden coffer-dams as in Cases 1 and 2—these caissons to be sunk to the same depth as the others and to have the entire working chambers of same filled with Louisville cement concrete, the remainder of the caissons to be filled with sand. Excavation between caissons and piers same as in Cases 1 and 2. This case presents some advantages and would perhaps be preferable to the wooden caissons, but for the fact that it may be practically impossible at the present time to get the plates necessary to construct these caissons of steel quickly enough to insure the work being done within the low-water period in the river. I have only considered this case within the past 24 hours, and am therefore unable to submit you a complete drawing of the steel caissons, but hand you herewith a pencil sketch which will serve the purpose of showing the construction contemplated. This caisson being only 6 ft. in width at the bottom, while the wooden caisson is 12 by 13 ft., it will serve the same purpose so far as shutting off the outflow of material from underneath the old pier is concerned, but not having so wide a base, it has been considered best to fill the entire working chamber of these caissons with concrete, whereas in the other case, with the wider base, we can probably, with perfect safety, use part concrete and part sand as described in Case 2.

"Pier 2, as above described..................$30 770.
Pier 4, as above described.................. 26 655.

"In connection with this work, I will furnish and put in place the concrete filling required for the spaces between the new caissons and the old piers, at the following prices per yard:

"For Louisville cement concrete..............$4.85
For Portland cement concrete.............. 7.38

"In all of the concrete work herein contemplated, the proportions for mixing are assumed at 1, 2, and 4 for Louisville cement concrete, and 1, 3, and 6 for imported Portland cement concrete. The quality of Portland cement assumed to be equal to Alsen's German.

"In this connection, I wish to say that we have a plant on hand ready to do this work, and could commence it immediately, our plant being now idle at Jefferson City, Mo. I will be glad to do any extra

work that may be required, not covered by these proposals, on a basis of cost plus 10%, including, of course, the equipment costs as well as labor costs.

"For the purpose of comparison, and illustrating the expensive character of the repair work contemplated on these two piers of your bridge, will say that I should be glad to furnish all material and build an entire new set of piers for that bridge according to the best modern specifications and practice, for the sum of $125 000.

"Very respectfully submitted,

"A. J. Tullock."

"Leavenworth, Kans., Aug. 18, 1898.

"Mr. E. Fisher,
"Engineer, B. & B., Mo. Pac. Ry.,
"Pacific, Mo.

"Dear Sir: Referring to the caissons for the repair work of your piers at Little Rock, we figure the cost for sinking the extra depth on these caissons, assuming that they are to go through bed-rock, at about $800 per foot for the large caissons for Pier 2 and about $750 per foot for the smaller caissons Pier 4. We assume, of course, that if this extra depth is required, that the excavation will be practically all solid rock, which makes the cost run up quite high. I have to-day carefully examined the record of the borings made by ourselves a few years ago, and also those made as reported in your specifications, and I feel quite certain that the depth you have specified will prove to be about right, and that no extra depth will be required. Certainly none will be required, unless we should strike a fissure in the rock, which it would be necessary to shut off, and even then in that event, we could probably clean it out and plug it with concrete inside the caisson much cheaper than sinking the whole caisson farther down. In figuring the cost of the entire work, we have distributed the plant charge, machinery, repairs, maintenance, use of plant, and all charges of that kind over the depth specified, and as this charge would remain the same, or practically the same, for a less depth, the saving for sinking less than the depth specified would not be a great deal, particularly if we assume that down to that depth only a portion of the excavation is rock. We figure, therefore, that the saving, in case we do not go as deep as specified, would be about half the price named per foot for extra depth.

"The price which we name for extra depth would only apply for a depth of 5 ft. below the depth specified in your specification. For any further depth, there will be an increase of about 50 per cent. This increase for extra depth is partly due to the fact that the schedule of wages paid the pressure men or 'sand hog' increases for every foot below 20 ft.

"In reference to completion of the work, we would expect to complete the entire work in four months, providing we receive the order for it immediately, and I will undertake to do it in that time, barring accidents or floods or any such things as would be beyond the control of the Contractor. I feel certain, however, that nothing is likely to occur that would stand in the way of the entire completion of the work in about four months, and we have made our figures on that basis. If it took much longer than that time, we would be the principal losers, as the cost would increase very fast by the extra time used.

"In going over the figures on this work to-day carefully, I find that we have made them exceedingly close, and if we do that work, we should want to know at once about it, or within the next few days at farthest. I shall be glad, therefore, if you will reach a conclusion as soon as possible and notify us.

"Yours truly,

"A. J. TULLOCK.

"Proprietor."

"LEAVENWORTH, KANS., Aug. 19, 1898.

"MR. E. FISHER,
"Engineer, B. & B., Mo. Pac. Ry.,
"Pacific, Mo.

"DEAR SIR: I have your favor of the 17th inst. with reference to modifying the dimensions on caissons for repairing the piers of your Little Rock Bridge, and have carefully noted what you say. I hardly think that it would do to figure on reducing the size of these caissons, nor do I think it would be desirable to do so. In the first place, we have no greater width between the caisson and the pier than is necessary to properly support the pier. In the second place, we cannot very well work closer to the old pier safely than 6 ft., as by getting much closer we would be likely to encounter more serious difficulties than are now contemplated and add to the cost. Further, in caissons of this character, on account of being so unwieldy and difficult to handle, it is not likely that they can be kept in exact position; in other words, the caisson is liable to be from 6 to 12 in. out of position in almost any direction, and this perhaps cannot be avoided. Of course, we will endeavor to keep it in position, or if out of position at all, to have the greatest distance between caisson and pier on the side to which the pier leans, so as to have the heaviest supporting wall on that side. We might save some caisson material by cutting down the dimensions of the caissons a little, but we would at the same time, increase the cost per unit quantity in certain parts of the work, so that on the whole there would not be much saving. It will be a difficult job at best to keep the wooden caissons of that character from racking to pieces on account of uneven support, even with a width of 12 or 13 ft., and

the more we reduce its width the greater this difficulty becomes. So far as the caisson itself is concerned, and as to the opening between the caisson and the old pier, if we get much closer to the pier than we have figured, we will increase the cost of sinking more than the saving made in caisson material by reducing its exterior dimensions.

"In the case of the Kansas City draw-pier, where this plan was followed with a circular caisson constructed of steel, the caisson was brought closer to the pier than in your case, being only 3 or 4 ft. from it, but in sinking that caisson, the average progress per day was from 3 to 5 in. for each day of 24 hours, and the entire cost for that pier was between \$40 000 and \$50 000, being somewhere close to the latter sum. We have studied the subject quite thoroughly and carefully from an engineering standpoint, and also from a practical standpoint with Foreman Stewart, who has had large experience in such work, and it seems to me that we have hit upon the best construction for the place at the least possible cost. I realize fully that this is a good deal of money to put on repairing two old piers, but I do not see any way to better the situation unless you should decide to put in entire new piers throughout the whole bridge, and in answer to your inquiries about that project, will say that the price I gave you of \$125 000 for replacing the entire piers contemplated putting the further pier on the Argenta side on piles, which could probably be safely done. However, we could make that pier a pneumatic pier also putting it down to rock, by increasing the cost to \$135 000, or thereabouts, and of course in these new piers we would not expect to use Cabin Creek stone except for backing, the face stone to be strictly first-class masonry, both in quality of stone and workmanship, instead of cheap rubble, as in the case with the present old piers of that bridge.

"I find that I shall probably have to be in Chicago on Monday on some bridge business for the Santa Fe, but can be in St. Louis Tuesday, or if you wish to take up the subject Monday and could wire me to-morrow, I might postpone my Chicago business until Tuesday. Should you decide to have us do this work, I would thank you to have Mr. Spoor, your Timber Agent, give me the names and addresses of the mills which could furnish this timber quickest and best, as it would be necessary to move at once in the matter of getting timber and equipment on the ground in order to be sure of getting the work done before there is danger of high water.

"Very truly yours,

"A. J. TULLOCK,

"Proprietor."

APPENDIX C

LETTERS OF A. J. TULLOCK, WHILE CARRYING OUT WORK RECOMMENDED BY PATTON.

"ST. LOUIS, November 24, 1898.

"E. FISHER, ESQ.,
"Engineer, B. & B., Missouri Pacific Railway,
"Pacific, Missouri.

"DEAR SIR: We have the caisson of the center pier built up about 8 or 10 ft., and last night we placed the second course of decking on the caisson. It is already lowered, so that one-third of its weight is taken up by water displacement, thus relieving the lower screws and supporting piles. We are, therefore, safely past one state of danger, which arose from the fact that our piles did not have much penetration and being difficult to brace. The matter of supporting so large a caisson on lowering screws was naturally a deal of a problem and somewhat risky. We feel, however, that we are now safely past any danger from the trestle support, and we shall, within the next two days, unless something unexpected interferes, have the caisson resting entirely on water and the screws relieved and removed to Pier 4. As soon as the screws can be set up on Pier 4, we will proceed with the building of the caisson, while we continue the sinking of the caisson at Pier 2. As near as I can tell, we should have air in the caisson of Pier 2 in about ten days or two weeks at the farthest. This, however, depending somewhat upon how much trouble we have in getting a proper bearing for this large caisson on the bed of the river. You will readily understand from the shape of this caisson in its unusual dimensions, that it becomes very necessary to handle it gently and lay it on the bottom in a good and reasonable uniform support, in order to prevent it from being strained and the joints opened so that it will not serve its purpose. Very little irregularity of support when the caisson commences to be loaded would likely spring it badly, notwithstanding the fact that we have built it unusually strong. It, therefore, becomes very important to get it carefully landed and this is our next critical movement. The fact that the sand has all been scoured away from that pier, which we did not expect, increases this difficulty very greatly, and it is impossible to make a fill around the pier by wing-dams above, as there is not enough material carried by the water at present to make such a deposit practicable. We have to depend, therefore, upon leveling off as much as we can before resting the caisson on the bottom, and then using sacks of sand for building up underneath the cutting edge of the caisson for support at the low points, and with this in view, I have already shipped several thousand coffee sacks from St. Louis to Little Rock so as to be in readiness. Superintendent

Stewart has the work well in hand and understands thoroughly every move to be made and every emergency likely to be met with.

"We are now approaching a time on this work when we must think carefully about what you are going to do when these caissons are down in position and the excavations made between the new caissons and the old piers. Of course, you cannot tell exactly until these excavations are made, but you must contemplate to some extent in advance, what material will be required, so that this material, such as Louisville cement and crushed rock, can be procured, on hand ready for use. You will recollect that while our contract provides a price for the concrete filling of this area between the new caisson and the old pier, the work of so doing is left for later determination. Therefore, I have provided no materials for such filling, and can only do so on your order. I would suggest that you authorize me to procure a certain amount of Louisville cement and crushed rock, which you think will safely be needed, immediately, so as to have it on hand when we are ready for it. You can certainly approximate the minimum quantity needed, and it is more than likely that you will want to fill this whole area as far as possible with concrete, in order to get the best results. Now, while it is important for the Contractor, on account of the coffer-dam and caisson and danger of floods in the river, that this work should be dispatched very rapidly after the excavation is made, it is still more important for the Railway Company and the owners of the old bridge that the structure should not be left exposed a *single hour longer than is absolutely necessary after this excavation is made.* There are abundant reasons for this, which you will doubtless recognize at once, and I now only mention one or two of them. In the first place, that old pier, caisson, and crib was so poorly constructed at the beginning that even if it were in as good condition as when first built, it would be very bad judgment to allow it to remain any length of time with the material all excavated away from it, as we are required to excavate. There is always danger in such performance of something of the unexpected happening, even if the pier stands perfectly square and true and is well built. Again, the timbers in that old caisson and crib, which are rough hewn timbers, improperly framed and about half enough in quantity, have now been soaking in the river about 14 or 15 years, and there is no telling what effect exposure to the air, for even two or three days, might have on these old timbers, heavily loaded as they are, without being reinforced by any interior filling of concrete or any other material of value. It has probably come to your notice that certain kinds of timber long submerged in water when taken to the surface, apparently in excellent condition, come to pieces very quickly after exposure to the air. I do not know that there is any danger to the timbers of these old piers, but at the same time it is possible that there may be and it is a matter

worth thinking about. So on the whole, it is undoubtedly the best policy to be ready to act quickly when the excavation is made, and to get the filling with concrete made around the old pier as high as possible and within the shortest possible time after the excavation is made.

"In the case of Pier 4, I have a suspicion that that pier is already taking a good deal of its lateral support from the material, sand, etc., which now surrounds the old crib and caisson. It is possible that it may be held directly in position by this material, which you will notice extends up quite high in that pier. This being the case, we will have to proceed very carefully indeed with our excavation, both in sinking the caisson and in making the excavation between the new caisson and the old pier, as that portion will necessarily be attended with a great deal of danger, and conditions may be discovered as the work progresses, particularly with reference to the excavation between the caisson and the pier, which would make it necessary to change your plan of operations materially. I mention this now only that you may be thinking about it in advance, as I have been for some time past.

"In this connection, you will see at once the necessity of having very frequent observations taken as to the movements of these piers from the time we commence excavating, even in the new caissons alone, until we are through with the work. It is altogether likely that at certain stages of the work, it will be necessary to take these observations several times a day, and even almost hourly, so that any movement whatever of the old piers will be detected and made known to our men in charge. I mention this now so you will make arrangements with your Assistant Engineer, who has been taking these observations, to be present on the work when our excavations commence, ready to report promptly any appreciable movement of the piers. I do not anticipate any movement of consequence at the center pier (Pier 2) but of course that depends on what we may find the conditions to be underneath. I have a young office engineer, Mr. St. John, on the ground, who has experience in the use of field instruments, and I shall be glad to have him assist your engineer in taking these observations when the time arrives. Please let me know in good time if you will arrange to have your engineer present to look after this matter during our excavations. It is important that this matter should receive careful and accurate attention. Write me at Leavenworth.

"Very truly yours,

"A. J. Tullock."

"Mr. E. Fisher, "May 16, 1899.

"Engineer, B. & B., Missouri Pacific Railway,

"Pacific, Mo.

"Dear Sir: Your favor of May 6th, written at Little Rock, Ark., was duly received during my late absence. I note your instructions in

reference to the work remaining to be done at Pier 4, that is, the concrete filling, etc. I believe the plan which you propose, which is substantially that adopted at Pier 2, is a very good one, and will get the best results with the least possible expense. I think the idea of supporting the old crib as high up as possible with concrete, as we have done at Pier 2, is the best solution of this difficulty, and I also believe that the material below the concrete, if of a reasonably substantial character, such as sand or rip-rap, is equally good when covered with a sufficient amount of concrete on top, which permits you to give the desired support to the old crib and pier, with the use of a minimum amount of concrete, much less in fact, than was originally contemplated. We will, therefore, probably have considerable extra cement left on hand, which your Company can no doubt dispose of or use to advantage.

"I note what you say in regard to pumping down and excavating between the coffer-dam and the old crib, and in reply will say that I will be glad to pump down and excavate just so far as it is practicable to do, but cannot undertake to go farther. This is precisely the conditions of our contract, and you no doubt fully realize that any attempt to do more than is practicable to be done in this case, would not only be a needless waste, not contemplated, but might also, particularly in the case of Pier 4, greatly endanger that pier. You will recollect further that when you had plans and specifications prepared for this work by some eastern professor or school teacher, your specifications provided for doing a whole lot of theoretical experimental work which was absolutely absurd, and which could not possibly have been carried out under any condition. I called your attention to this at the time, and I think that you agreed with me at that time in reference to it, and I suggested the present plan of work in my proposal, which was adopted. When the contract was drawn by your attorney, which was done during my absence and afterward presented to me for signature, I called attention to the fact that in drawing it, your attorney had provided for what was probably impossible, explaining that it would probably be impracticable to do certain work which seemed to be contemplated in the contract, the character of which was somewhat the same as you had in the professor's original specifications. Your attorney then modified his language in the contract by adopting my own expression in regard to pumping and excavating, which was that we should do just so much as was practicable to be done. Now this is the meaning of our contract, and I wish to make it clear to you, as you seem in your letter to want to swing a club over the head of your contractor, by talking about compelling him to pump down below the cutting edge of the old crib and caisson and do certain other impossible and absurd things, and you know very well that there is no object whatever in doing this, even if

it could be done, and if that material, which is in this case mostly all rip-rap stone, could be taken out, you would simply have to put it back again to give the necessary support to the pier or to substitute concrete in its place, which would cost your company several thousand dollars more than the present arrangement and would be practically no better. Furthermore, you must be aware that the inclination of Pier 4 is so severe to one side that it would be extremely dangerous and inviting disaster to excavate that material between the new caisson and the old crib to any great depth. This is all clearly manifest. I am writing you thus fully as I wish to make my views clear on the subject and wish to put them on record. In sinking the caisson around Pier 4 we went down as you know on all sides through rip-rap most of the way. There was a little less rip-rap on the down-stream side of the pier than on the up-stream side, but nearly all the filling, at least, the main mass of the material around the old crib and between it and the coffer-dam, is now rip-rap, and by putting a good solid bed of concrete on top of it, you will have supported that pier in the best possible manner, as with the new caisson in position it cannot get away.

"I dislike very much to complain, but I must say that the whole tenor of your demands and instructions seems to be of the nature of threats of putting your contractor to unwarranted costs and expense rather than for the purpose of getting the best results on work or of giving any special benefit to the job or your company. You seem to lose sight entirely of the dangerous and costly nature of the work we have to perform around these two piers successfully, and of the great cost and heavy loss which has been sustained in carrying out this work safely. You seem to be looking only for a chance to put your contractor to needless and foolish expense of one or two hundred dollars in doing something which you imagine you have a right to demand, whether it is any value to the work or not, and to entirely ignore the fact that in sinking these tremendous large caissons around your old piers through rip-rap, piles and everything else, and through a large amount of solid rock, we have been compelled to proceed so slowly in order to avoid injuring your old piers, that the cost of doing the work has been double what was contemplated.

"I note that you say in your letter 'before concrete is put in, the depth of the cutting edge must be determined.' Now, when we have pumped down between the coffer-dam and the old crib of Pier 4 and have gotten that crib ready for concreting, we must be ready to concrete at once and finish quickly, as it will be dangerous and absurd to endeavor to hold that pumped out coffer-dam against chances of rises in the river and other disasters any length of time, for a foolish effort to try and determine the depth of the cutting edge of the old caisson. At best, the determining of the elevation of this cutting edge is only to gratify a curiosity and can serve no purpose.

Whatever its location might be found to be, if possible to find it, it would not in any way affect the character of the work to be done, the treatment of the pier would remain the same. Furthermore, as you must be aware, Pier 4 is surrounded with rip-rap, and in sinking our caissons around that pier we went through a solid mass of rip-rap stone nearly all the way down; consequently, it will be practically impossible to get a rod or a hook down the side of the old crib, so as to reach the cutting edge of the old caisson underneath it. This I believe was finally accomplished with some uncertainty at Pier 2, but in my judgment it will be entirely impossible at Pier 4, and it will be very foolish, after the coffer-dam is pumped out and we are ready to concrete, to waste any time or assume any risk in any such idle experiments. You may safely take it for granted, that wherever the cutting edge of that old pier is, the pier is settling and leaning dangerously, and must have the support which we propose putting in there, and that just as quickly as possible when the water is pumped out.

"Answering your question directly as to hunting for this cutting edge, will say that, of course, if your company will assume all responsibility and all costs, direct and indirect, and all consequent damages, which might occur from floods and other disasters, by delaying the putting in of that concrete around Pier 4, after we are ready for it, then, of course, we will hunt for the cutting edge of that caisson, at your expense, just as long as you want to continue such investigations, but when you think the matter over carefully, I think you will agree with me that no time should be lost, after we are ready to concrete, for the purpose of satisfying curiosity. I am thoroughly satisfied myself as to where the cutting edge is under that old pier, and when you see the top of the old crib, you can satisfy yourself beyond a doubt as to where the bottom is.

"Very truly yours,

"A. J. Tullock,

"Proprietor."

APPENDIX D

LETTERS OF RALPH MODJESKI CONTAINING RECOMMENDATIONS FOR REINFORCEMENT OF THE PIERS.

"CHICAGO, ILL., August 5, 1911.

"MR. E. J. PEARSON,
"*First Vice-President,*
"Missouri Pacific Railway Company,
"St. Louis, Missouri.

"DEAR MR. PEARSON:—As had been prearranged, I visited, last Thursday, in company with Mr. C. E. Smith, Bridge Engineer, your two bridges at Little Rock, Arkansas. Most of the time was devoted to the examination of your Little Rock Junction Bridge, which is in a precarious condition. I have also carefully examined the plans and records of the old structure, together with what has been done to repair it.

"It would seem that the coffer-dams which were placed by pneumatic process around Piers II and IV did not get at the seat of the trouble, and as shown by Pier IV did not correct it. The original caissons were sunk to rock and the air chamber filled with concrete, but the cribwork above was filled with sand and possibly some rip-rap. There is no doubt in my mind that the concrete in the caisson chamber rests on rock. It would be difficult to suppose that the engineers had come within a foot or two from rock and had failed to remove that thickness of material before sealing the chamber. The records as well as the tradition seem to indicate positively that the air chambers were sealed with concrete. There seems to be no doubt therefore that the trouble occurs in the cribwork above, which, instead of being filled with concrete as it should have been, was filled with sand and possibly some rip-rap. This sand is probably leaking out through the openings in the outside sheeting and is settling away from the timbers, so that the entire load is carried on those timbers, which under a load of something like four or five hundred pounds per square inch are crushing.

"I should like to have one or two more days to consider this whole matter carefully, and will therefore not make this my final report as to what is best to be done. I have no doubt, however, that what we should attempt to do is to remove the sand from the inside of the cribs and replace it with concrete. If that is done to all the piers it would afford excellent foundations for future piers which you may wish to construct for a new bridge or else it would support the present piers as long as the piers themselves held together.

"Pier III is the one which should receive immediate attention as it is gradually settling at a rate which in a few months time

would mean its total destruction. In conference with Mr. Smith it was agreed that a plant be rigged up on a barge for taking wash-borings around the pier to find out if there is sufficient rip-rap present to prevent driving a wooden triple-lap sheet-piling coffer-dam. It is also desirable to arrange as soon as possible to bore at least two holes in the up-stream corners of the coffer-dam to find just how much material is left inside of it, and also incidentally to find whether the working chamber has actually been sealed with concrete.

"My present plan would be to drive a coffer-dam around the pier at a distance of say 6 ft., from the walls of the old crib and to pump it out, lowering the water level sufficiently to examine the old crib by partly removing the outside planking. While this is being done it would be quite advisable to partly support the up-stream end of Pier 3 by shores which in their turn would rest on piles driven alongside the old coffer-dam. Mr. Smith is familiar with what I have in mind.

"Whatever work is done, it will have to be performed carefully and step by step, in order not to endanger the present pier. It is not possible to foresee all the difficulties that may be encountered, but I am quite confident that the work can be done at a reasonable cost.

"I would recommend that the work be done by Company forces, thus saving considerable time and possibly expense as well.

"I expect to write you again Monday, after studying the situation a little further.

"Very truly yours,

"Ralph Modjeski."

"Chicago, August 7th, 1911.

"Mr. C. E. Smith,

"*Bridge Engineer,*

"Missouri Pacific Railway,

"St. Louis, Mo.

"Dear Mr. Smith.—I have been thinking a great deal about the Little Rock proposition since I saw you at St. Louis, and I cannot arrive at any more satisfactory solution than the one which I outlined. There is another possibility, however, which occurred to me, and that is that it may not be necessary to drive a coffer-dam around the pier at 6 ft. distance from the old coffer-dam. The farther away we go the more difficult it would be to keep the water out during pumping. I would suggest, therefore, that, as contemplated, you take the soundings and borings to ascertain how much rip-rap there is around the pier, and then either simultaneously or immediately thereafter arrange to bore a hole through the corners of the cribs vertically to find out how much sand or rip-rap or other material there is in the crib. It may be that if the material contained in these

cribs is merely sand or such material as can be pumped out, it will be sufficient to drive a Wakefield sheet-pile coffer-dam at a distance of 1 ft., for instance, in which case it would be very easy to excavate the material between the old crib and the coffer-dam and seal the bottom with concrete. It would take very little concrete and of little thickness to hold water. After this is done we could pump out the coffer-dam sufficiently to get at the top timbers of the crib. We could then remove these timbers at least partly and remove as much as possible of the material inside. It is probable that we might be able to pump out the water from the interior of the crib and then fill it with concrete in the dry.

"Very truly yours,

"RALPH MODJESKI."

"CHICAGO, August 19, 1911.

"MR. E. J. PEARSON,
"Vice-President, Missouri Pacific Railway,
"St. Louis, Mo.

"DEAR MR. PEARSON: Since writing you the report on the Little Rock Junction Bridge, I have been thinking the matter over, and it occurs to me that if the Company desires to maintain this bridge for some years to come and perhaps build a new superstructure on it, it would be cheaper in the end to rebuild the piers of concrete, at least Piers 3 and 4, which are causing the most trouble. Pier 3 is in a precarious condition, and no method of repairing, unless it be a very expensive one, can be counted on with absolute certainty to produce satisfactory results, and at best the masonry in the present piers is not in the very best condition, many of the stones being cracked, and while it may last for many years, it would have to be replaced some time or other. I would respectfully suggest that you authorize the reconstruction of Piers 3 and 4 at this time. In this way the masonry can be taken down and the cribs filled solid with concrete at much less expense than could be done with the piers in place. The concrete piers could be built for a reasonable amount and part of that cost would be offset by the saving in the cost of the foundation repairs, which repairs, as stated, would be rendered considerably easier.

"Kindly advise me what your wishes are in this respect.

"Very truly yours,

"RALPH MODJESKI."

APPENDIX E

List of Equipment Used in Repairs to Pier 3.

1 barge, 24 by 80 ft., with derrick.
1 barge, 18 by 60 ft.
1 barge, 16 by 50 ft.
1 barge, 15 by 24 ft.
1 rowboat.
1 compressor, 18 by 18¼ by 24 in., with receiver and mains.
2 air locks.
2 hoisting engines.
3 horizontal boilers.
1 derrick.
1 orange-peel dredge, ¾-yd.
1 pile leads and hammer (not used).
1 No. 4 Emerson pump.
1 pulsometer.
1 12 by 7 by 12 Fairbanks-Morse pump.
1 D. C. centrifugal pump, 8-in.
2 steam jets.
1 diaphragm pump.
1 double-drum crab.
1 No. 11 Smith mixer.
4 steel concrete cars.
1 turntable.
6 steel concrete buckets.
1 steel tremie.

DISCUSSION

HENRY H. QUIMBY,* M. AM. SOC. C. E.—It will probably be of interest to mention a timber crib pier foundation which is not solid, is more than 50 years old, is still doing its work satisfactorily, and probably will do so as long as it is wanted. The conclusions of the author are that timber cribbing is not suitable for bridge foundations unless it is solid. Mr. Quimby.

Chestnut Street Bridge, over the Schuylkill River in Philadelphia, was built during the Civil War. The foundations of the pier in the middle of the river were laid in 1862. A crib of hewed square yellow pine, 29 ft. high, extended from the bed of the river to a point 2½ ft. below ordinary low water. The overlying gravel was dredged from the rock, and measurements were made. Then the crib was built at the side of the river to fit the contour of the rock, and towed to place and sunk with stone and gravel screenings in all the coffers. The masonry work was then commenced.

The reason that crib is doing its work satisfactorily is that it was built in a very different way from that used in the cribs described in the paper. The cross-timbers, spaced so as to leave coffers about 3 ft. square, were notched over each other at their intersections. Each stick was notched in one-quarter on each side, so that the timbers were theoretically in bearing throughout their length. The structure settled as the masonry superstructure of the pier was built on it, and this settlement was a little unequal, causing a slight list to one side. The total settlement is said to have been 6½ in. in the depth of 29 ft., and could be accounted for in part by the inaccuracy of the fit of the notches, and in part by the compression of the timber, the roughnesses of the hewed sides probably causing much of it.

A rough calculation, making guesses as to the total load that the crib is carrying, and the quantity of timber it contains, indicates that the compressive stress in the timber is less than 100 lb. per sq. in. The average pressure on the crib described in the paper, according to the author's calculation, and supposing the load to be central, is about 400 lb. per sq. in. of intersection bearing. The drawings show the timbers merely resting on each other at the intersections, and not notched over. The eccentricity of the load, as computed by the author, however, brings the maximum pressure up to 880 lb. per sq. in., which is entirely too much for side-grain bearing, and too much for end-grain for a continuing load, even on dry wood.

Wood appears to flow under pressure, the same as steel, and as concrete is found to flow. Therefore a long-continued pressure ought not to be as great as that of a transitory load. Bridge men have been brought up to regard live-load as more serious than dead-load stresses, but the reverse may really be the case, because a continued stress far

* Philadelphia, Pa.

Mr. Quimby. below the rated ultimate strength may cause flow, and, as in the case of the Quebec Bridge, ultimate failure.

In a recent case of falsework, the vertical members were stressed, in some instances, up to 1 000 lb. per sq. in., and where the ends of the verticals bore on the side-grain of the sills and caps they crushed into it. The sills were blocked up, where adjustment was intended to be made, on blocks twice as long as they were wide, reducing the pressure on the blocks to one-half, or 500 lb. per sq. in., across the grain. The wood was saturated with water, the load was long continued, and quite a number of the blocks crushed. On some of them the sapwood split away from the heartwood, seemingly by reason of its swelling, under the action of the water; not being confined laterally, as it was vertically, it swelled in this direction as the blocks crushed.

The wood in the cribs described in the paper was evidently overloaded, and it simply crushed under the weight. The only wonder is that the engineers were not more alarmed about the condition of those piers. The illustrations show that it was very bad.

The most perplexing problems that structural engineers have to meet are generally found in such jobs as this—the repairing or strengthening of existing structures. The problem was met very successfully by the final strengthening.

Not the least interesting thing in this paper is the statement that the bridge superstructure, built 30 years ago, presumably for loads common in that day, is good for modern loads, and will be for many years to come. The probability is that that bridge, 30 years old, is of iron, not steel. Iron, in those days, was customarily stressed lower, in proportion to its ultimate strength, than steel is to-day, which accounts for the continued service of many old iron structures, including some of the elevated railways, under loads several times as heavy as were contemplated in the design.

Mr. Byers. M. L. BYERS,* M. AM. SOC. C. E. (by letter).—The object of this paper, as stated by the author, is praiseworthy, and in connection with the subject there are quite a number of points of interest to the Engineering Profession.

The writer was surprised, however, to find in the paper several of his signed official communications. As these communications are separated from their context, they are, perhaps, somewhat subject to misinterpretation. For example, the communication commencing on page 42, which is given in part, seems to require, for a complete understanding of the reasons governing its character, the knowledge that there was contained therein, among the various matters submitted, certain other tentative plans for general improvement in the vicinity which contemplated the entire abandonment of this bridge, and, before

* New York City.

the final decision could be made as to the engineering features of the pier situation, it was necessary that these general development plans be finally passed upon by the management. Mr. Byers.

With reference to the effect of the temporary treatment of Pier 4 which was resorted to in 1908, the author seems to have changed his mind, as he states, on page 34:

"Following this work, no appreciable movement in the direction of the bridge was apparent for several months, but the movement at right angles to the bridge appeared to increase its rate, indicating that the work had no appreciable effect."

Whereas, in his memorandum to his superior, as of August 17th, 1909, which he has seen fit to quote on page 37, he states, "If the pier continues stationary, no action whatever is necessary, as the piers are all perfectly safe."

With reference to the yokes, the writer authorized their use with the express condition that they be constructed of such light section that it would be impossible for them to exert sufficient additional stress on the bridge members to be of vital importance.

It is a source of considerable satisfaction to the writer that, due to the successful temporizing with this problem for a number of years, during and immediately following the 1907 panic, when money was scarce and saving imperative, and at the expense of a very considerable amount of worry to himself and staff, the opportunity has been afforded to the present management to solve this difficult problem successfully and economically.

LEE HIGHLEY,* ASSOC. M. AM. SOC. C. E. (by letter).—Mr. Smith has given a very lucid and complete history of the trouble at the Little Rock Junction Bridge and its final correction. The writer obtained his first knowledge of the leading facts concerning this structure while working in a minor position in the Bridge and Building Department of the Missouri Pacific-Iron Mountain System. In fact, it sometimes fell to his lot to make the measurements for finding the movement of Pier 4. The earlier measurements were comparatively simple and devoid of elaboration. They took into account only the movement of Piers 3, 4, and 5 in the direction of the center line of the track and there was no determination of movement at right angles to this direction. From a fixed point in the center of the track behind the south abutment, called Pier 1, direct tape measurements were made along and over the ties to Piers 3, 4, and 5, using a plumb-bob from the top of the ties to chisel marks on the coping stones. A 15-lb. pull was put on the tape, and temperature allowance was made in calculating its length. It happened sometimes that the ties were wet from rain or possibly covered by a light fall of melting snow, and thus the tem- Mr. Highley.

* New Meadows, Idaho.

Mr. Highley. perature of practically one-half of the tape might differ considerably from that of the other half which was not in contact with the ties. In all cases where there was a suspicious difference between the results of any measurement as compared with previous ones the work was re-checked. The character of the weather, direction of the wind, and gauge reading of the river were also recorded. Despite the crude features, there was no getting away from the fact that Pier 4 was steadily moving in the direction it was leaning, through some periods slowly, through others with more acceleration.

Time went on, and conditions grew worse. In 1908, the observations included additional features. A system of cross transit lines, intersecting on the up-stream end of Pier 4, was used in addition to the straight tape measurement. Gas-pipe wood-filled hubs, with brass screw centers, were sunk flush with the surface of the earth about 600 ft. up and down-stream from the Argenta or north end of the bridge. From these points a transit was sighted to fixed objects, corners of brick buildings, on the Little Rock side of the river. At the intersection of the transit lines a copper plug was fixed in the coping of the pier, and properly marked. From this intersection point lines were laid out at right angles and marked on the coping. One line was parallel to the center of the track and the other at an angle of 90° thereto. As the pier continued to move and future observations were taken, the point of intersection would necessarily fall within the quadrant determined by the two lines first laid down. By measuring the offset from the new intersection to the old lines, the movement of the top of the pier in two directions was determined. However, the transit observations, following this method, were not always satisfactory, were sometimes attended by difficult physical conditions, and did not always check reasonably close with the tape measurement. It should be remembered that the fore-sight objects were nearly half a mile away, across the river diagonally, and that fog and smoke often rendered it difficult to see. Also, as Mr. Smith points out, the continued movement of the pier soon brought bridge members across the line of sight and added to the complications. If the fixed time for observation fell on a dark, cloudy day it became necessary to use lights and reflectors to illuminate the plumb-bob cord, or small rod, which was used for obtaining points on the pier. The usual custom was to set three times on the fore-sight and give as many sets of points for determining lines, then take the mean of the three for the intersection line. Even under the most favorable light conditions, it was the writer's practice to take advantage of the sun's position, and lay one line of intersection in the morning and the other in the afternoon in order to get the truest reflection of light from the fixed fore-sight and the one on the pier.

The writer left the service of the Missouri Pacific System early in 1910, and has been out of touch with matters concerning the Little

Rock Junction Bridge since that time. Therefore he finds this paper of unusual interest. The newer method of observing pier movements, as shown by Fig. 33, is much superior to those formerly used. Mr. Highley.

The plan of procedure for depositing concrete in the crushed end and side of the old timber cribs as the excavation proceeded, thus causing the leaning piers to right themselves to a considerable extent, is regarded as particularly ingenious. On the whole, it is gratifying to know that Mr. Smith has terminated so successfully the work of transforming the unsteady old piers into substantial, permanent, and symmetrical structures.

THEODORE BELZNER,* ASSOC. AM. SOC. C. E. (by letter).—In his report to Mr. Fisher, Mr. Patton suggested that the material between the bottom of the caisson and the rock might be solidified by "injecting pure (liquid) cement under pressure into the material, thereby converting it into concrete in place". If Mr. Patton or the author made any experiments with this process, it would be interesting to know the results. The writer recalls a few experiments, made some years ago, in New York City, by forcing grout into sand under pressure. When the material into which the grout had been pumped was removed, it was found that the cement had not been distributed uniformly. The material was lumpy and of no value whatever. Experiments with this process by other engineers may have produced different results. Mr. Belzner.

ROBERT H. P. FORD,† ASSOC. M. AM. SOC. C. E. (by letter).—Mr. Smith has contributed an important and interesting paper, which should prove of value to engineers having to deal with foundations and piers, especially in the large streams throughout the South and Southwest, practically all of which carry large quantities of sediment, and are subject to an erratic rise and fall combined with excessive scour, giving opportunities for skill and resourcefulness in work of construction or repair. Mr. Ford.

The writer was connected with the Engineering Department of the Missouri Pacific System during the period when the steel bent referred to by the author was added to Pier 4, or the "Sick Pier", as it was popularly known, and he speaks from a somewhat personal knowledge of the work that Mr. Smith has done, first as a subordinate and afterward in direct charge of reconstruction, which has been both ingenious and skillful, demonstrating resourcefulness as well as engineering ability of a high order. The writer feels, however, that Mr. Smith has perhaps unwittingly erred in his reference to Mr. C. D. Purdon, an able and accomplished engineer, who was Inspector during the original construction, as well as to others who at

* New York City.
† Chicago, Ill.

Mr. Ford. various times during the past 30 years have been connected with this work.

Early in 1908, the writer, under the direction of the Chief Engineer of Maintenance of Way, had occasion to take over, among other matters, the work at the Little Rock Junction Bridge, shortly after it had been determined to support the spans on Pier 4 by falsework, preparatory to taking down this pier. It was found at the time that very little information was available that would give a precise technical history of Pier 4, or in fact of any of this work, and, in the judgment of the writer, this was so essential, before a proper conception could be had of the problem, that immediate steps were taken for its collection. It was during this study that the writer became convinced that the authority of Mr. Purdon, as an Inspector, would not have enabled him to prevent unwise methods or improper work.

During the admirable work of Mr. Smith and his assistants, he has had available a fund of information and material which has been developed as a result of long study on this problem by numerous engineers and others, many of the former having recognized standing in the Profession, as well as the effect produced by the various expedients during the time that this bridge has been a source of apprehension; this has also contributed in no small degree to the information bearing on this problem.

Until very recently, the engineers' problem on this work has been to temporize with the existing conditions and prepare for an emergency, and, at the same time, to guard, if possible, against a direct failure. How well they have succeeded is perhaps best shown by the fact that nothing did happen, and traffic was maintained. Whether this was due to engineering skill, the forces of Nature, or good fortune, the reader must determine for himself; but to the mind of the writer, the characterization of the matter, up to the time of actual reconstruction, as a "farce comedy", is hardly fair to any of the numerous engineers who were directly or indirectly connected with this work during all this period.

The fact that a large expenditure was made previous to 1908 to correct this trouble, but which failed to accomplish its purpose, may perhaps be considered as inefficient engineering. The writer, however, hardly thinks this is the case. The records show that the work was done contrary to the recommendations of Mr. Tullock, although apparently approved by the engineers of the Company; but, back of this is the unfavorable comparison between the cost of the work by the method which was adopted and the cost for complete reconstruction, the course which it is believed all concerned would have very much preferred had funds been available.

The situation is not a new one. The Railway Company, like numerous other corporations, doubtless felt that the sum required to

enable it to take the safer course would be much greater than for that which seemed to be reasonably sure, although it contained a much greater element of chance. It is a well-known fact that, during the period when the conditions of this bridge were in a serious stage, traffic requirements on the Missouri Pacific System had out-distanced its physical condition to such an extent that the urgent demands for relief were not by any means confined to the Little Rock Junction Bridge. The financial condition of the property has not for years been such as to remedy a great many conditions which its management have at times felt should require attention. In any event, the records seem to show that it was beyond the power of its engineers at any time during this period to do what doubtless in their opinion should have been done, namely, to reconstruct the piers fully and finally, as was ultimately done under the direction of Mr. Smith. Mr. Ford.

The writer, though fully agreeing with Mr. Smith and his conclusions that, "When defects are discovered in bridge piers, and trouble results, correction should be applied at the seat of the trouble," differs however, with him in his deduction that, "The continuation of the trouble without adequate correction involves constant hazard, which can be avoided by efficient engineering talent."

The writer has every reason to believe that, had this matter been left solely to the judgment of the engineers connected with this work, even from its early inception, or during any of its later periods, the situation would never have been continued.

As is well known, the funds necessary for carrying out projects of this and like character are not always forthcoming, even though the engineer considers that they should be, and this case is no exception. Neither does the writer believe that, even with the numerous recurring changes of management on this property, which has had in its personnel a great many able and efficient men, who were in responsible charge of it, they were so derelict in their duty as to permit a condition to continue that could have been remedied if they had been able. He has reason to believe that the conditions were fully realized, although possibly not to the extent that the engineers may have felt on this as well as other important pieces of work, and, furthermore, that the situation prevailing on the Missouri Pacific in such matters did not differ from that on many other railway properties, where the advancing methods of railroading have caused the need for additional funds which have not always been forthcoming.

Mr. Smith's method of prosecuting the work, as well as his thorough study of the conditions preceding it, have doubtless resulted in saving the property considerable money. The writer, however, is inclined to agree with Mr. Modjeski, in his conclusions (given in Appendix D) that, given the problem as it was, his recommendations were justified. The element of chance in carrying out the work in

Mr. Ford. the manner in which it was done seems to have involved a greater risk, and one which the Railway Company was again willing to assume, thereby introducing an interesting comparison in this particular between the work performed previous to 1908, which did not accomplish its purpose, and the work of 1914, which did. Efficient construction and supervision, as well as good engineering, were the elements contributing to its successful conclusion.

Mr. Purdon. C. D. PURDON,* M. AM. SOC. C. E. (by letter).—The caissons referred to in the paper were located correctly, and the masonry fitted as well as it generally does, it being hardly practicable to sink a caisson in exact location.

When the rock was cut level, as soon as a part of the shoe of the caisson reached it, all the rest was cleaned out and blocked up with timber before placing the concrete.

The cribs were filled with rock to the top before any sand was pumped in, for the purpose of giving weight to the pier, and not with the intention of carrying the weight. It is possible that in time, with the shock of drift striking the piers, etc., the rock may have worked under the cross-timbers and caused a settlement.

The only trouble with the piers was that they were designed for spans 14 ft. wide, in the clear, and the masonry was well advanced before the superstructure plans were received; these showed that the spans were to be 18 ft. wide, in the clear, and it was necessary to widen and lengthen the tops of the piers to get a bearing. Even then it was a tight fit, but the spans fitted correctly on the piers. All the piers were practically complete before the steel for the superstructure was received.

The piers were all built in accordance with the plans and specifications, and the batter was the same on all, except that the pier at the north end of the draw-span was built with its south face vertical, to shorten the draw-span.

Any difference that may have been observed in the batter later is readily accounted for by the crushing of the timber in the cribs; because of the narrow base of the pier, a small settlement to one side would be largely multiplied at the top.

Mr. Jonah. F. G. JONAH,* M. AM. SOC. C. E. (by letter).—The writer has read Mr. Smith's valuable paper with interest, and from personal knowledge of the conditions believes that the work was handled with great skill and by the safest possible methods. The author was quite right in insisting on a plan which would obviate the use of falsework. To carry a span on falsework over the Arkansas River for any considerable length of time is a hazardous proposition. The records show that

* St. Louis, Mo.

serious rises have occurred in this river during every month in the year, and nearly every railroad which spans it has had trouble in the construction or renewal of bridges. Mr. Jonah.

One other plan might have been used in a case of this kind, and that would be to sink a cylinder at each end of the old pier, up and down stream, place a cross-girder between, and rest the ends of the spans on that, in which case the load could be taken from the old pier entirely. A defective pier in the Atchafalaya River Bridge, on the New Orleans, Texas and Mexico Lines, was handled in this way, but probably the plan pursued by Mr. Smith was, in the end, the best for the Arkansas River.

C. E. SMITH,* M. AM. SOC. C. E. (by letter).—Mr. Quimby mentions a very interesting case in which considerable settlement took place in a timber crib about two-thirds as high as the Little Rock crib and of much better construction. Had the Little Rock cribs been constructed similar to those described by Mr. Quimby, no apprehension would have been felt, regardless of the settlement. Mr. Smith.

The writer was very much surprised and filled with regret to learn that any one thought the paper reflected on the ability of any of those able engineers who had to do with the construction of the bridge or the correction of the trouble, as no reflection or criticism was intended, the paper having been written merely as a recital of facts.

Fortunately, full detailed information in the shape of original papers was available, and quotations were freely used where available to render the subject matter more accurate and to avoid any misunderstanding that might have arisen through digests of the quoted matter.

The writer believes that, with the knowledge at hand during the construction of the bridge and during the trying period of repairs, all those who had any connection whatever with the work handled the matter according to their best judgment, in conformity with the facts developed and resources available. Unfortunately, funds for better construction and for the proper correction of the trouble were not at all times available, but, had they been, there is no doubt that any one of the large number of able engineers connected with this problem would have corrected the difficulty in an entirely efficient and satisfactory manner long before the writer had any connection with it.

This is definitely shown in the following quotation from Mr. Purdon's letter in the original report, which indicates that he objected strenuously to the type of construction:

"I objected to this strongly at the time, but as I did not design the piers, this work being done by the late Mr. T. E. Sickels, I was not responsible for them. I told Mr. Wood at the time that I was satisfied

* St. Louis, Mo.

Mr. Smith. the timbers would eventually crush and put the piers out of shape, which it seems occurred."

In closing, the writer desires to express his appreciation to those who helped him in the final solution of this problem and who contributed so largely to the information now available; he wishes to express his regrets and apologies to any one who may have felt injured in any way by the paper as presented.

The association of the writer with all who have been connected with the work has been a source of great pleasure and benefit to him, and he has appreciated to the fullest extent the efficiency and integrity of those able engineers who struggled so faithfully with this problem, without the funds for proper solution.

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TRANSACTIONS

Paper No. 1336

FUNDAMENTAL PRINCIPLES OF PUBLIC UTILITY VALUATION*

By John W. Alvord, M. Am. Soc. C. E.

With Discussion by Messrs. Lewis M. Haupt, L. L. Jewel, Richard T. Dana, F. Lavis, Louis L. Tribus, Joseph Mayer, Philip Burgess, J. P. Newell, Charles Rufus Harte, Alexander C. Humphreys, H. F. Dunham, Stuart K. Knox, C. E. Grunsky, W. Kiersted, Clinton S. Burns, and John W. Alvord.

Synopsis.

This paper is undertaken as a consequence of the voluminous discussion of the writer's prior paper: "The Depreciation of Public Utility Properties as Affecting Their Valuation and Fair Return,"† presented December 17th, 1913. That discussion, though it showed the greatly increased interest which valuation matters are now receiving, also seemed to show that many engineers lack a clear perception of the fundamental principles involved in valuation work, presumably because such principles are to be found only in the law and the interpretation of the law by the Courts.

This paper shows that the science of valuation calls for a knowledge in three professions or callings: the law, engineering, and economics, and that, fundamentally, it is the law, and its interpretation by the higher Courts, which controls.

* Presented at the meeting of November 12th, 1914.

† *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 788.

The fundamental definitions of value and property are given, and the constitutional conception of what it is that is to be valued is shown, with citations from leading cases passed on by the Supreme Court.

The point is made that the law determines that it is a "property" rather than an actual past investment that is to be valued, and that the value must be found as of to-day.

The fact that we are to value a property as of to-day indicates why the reproduction method obtains its importance, but it is made clear that reproduction is only a cost method, that cost is not value, and that the Courts have indicated that all lines of material evidence should be examined.

An outline is given of the various lines of evidence that should be summed up, including evidence that cannot be expressed well in figures.

Reproduction is thus examined only as a line of cost evidence, and its conceptual requirements are discussed, illustrated, and summarized. Past cost is also discussed as a line of cost evidence, and its deficiencies in throwing light on value now are pointed out; misuse of cost evidence methods is explained, other lines of evidence not readily reduced to cost figures are described, and, finally, the importance of reasoning from cost lines of evidence—as well as other evidence—to value is emphasized and illustrated.

To summarize, it has been attempted by this paper to show:

First.—That it is quite idle for engineers, appraisers, utility commissions, and others to argue about the detailed methods of valuation unless they have a fundamental and uniform conception of what "value" and "property" are, in the first instance, and what the reasoning of the Courts has been on these subjects.

Second.—That, fundamentally, the art of valuation must rest in the law and the interpretation of the law by the Courts, and especially the Supreme Court of the United States, this being the final authority in valuation matters.

Third.—That the Supreme Court has repeatedly ruled that it is the value of the property used for the public purpose that must be considered, and it is the value of that property now.

Fourth.—That most of the utility commissions and many estimators who have been following their rulings are at variance with

the opinions and decisions of the Courts, in that they are insisting on retracing an investment rather than valuing a "property."

Fifth.—That the reproduction method is in most cases a most useful line of inquiry in arriving at a conclusion as to what the value of a property may be at the present time, but that it is not the only line of evidence to be considered.

Sixth.—That reproduction is a cost method and does not of itself determine value. It is only one of the means; not an end.

That the reproduction must be complete in itself, including going costs, before it can be properly used to reason from.

Seventh.—That when all lines of evidence are properly before us, we must reason as to what bearing on value each one of them may have.

A list of important legal decisions is appended to the paper.

INTRODUCTORY.

The problem of valuing the property of public utilities is a difficult one, largely on account of the fact that there is required a knowledge of certain fundamental principles of law, of engineering, and of economics or finance. No one of these professions has within itself all the training and information which must be brought to bear on the problem, in order to make an equitable valuation. Experience and information from all of them must be utilized to ensure sound conclusions.

The Courts are somewhat prone to criticize expert witnesses because they do not agree, forgetting often that the fundamental data are indeterminate; financiers and economists are impatient because judges and lawyers do not appreciate the natural laws of business and the reasoning of sound economics; and, finally the engineering specialist becomes irritated with the Courts because his views on such subjects as "property rights," "franchises," "past cost," and "present value" are not deferentially treated, little realizing that he is here speculating entirely within the domain of the legal profession, and is often guilty of attempting to make law, where much more competent and well-trained minds have long been at work in the endeavor to produce sound reasoning. The engineer is especially impatient because the economist sees no practical difficulties or uncertainties in construction, and both the economistic and legal type of mind are

too prone to regard the engineer as primarily a mathematician, who should produce exact results even from inexact data, little realizing that engineering work is often more judicial than mathematical in its ordinary application and practice.

It must be apparent that it is quite useless to endeavor to discuss intelligently the details of valuation work unless somewhere the underlying principles, both as regards engineering and the law and economics, are co-ordinated, and their relative value to the problem in hand is fully appreciated. It is the purpose of this paper to attempt this somewhat ambitious task.

Definition of Terms as Used in this Paper.

In the present state of confusion as to valuation nomenclature, it is desirable to define the meaning of the more important terms used in this paper, as follows:

Property.—That which is owned; that which belongs exclusively to an individual; that to which a person has a legal title (whether in his possession or not); the exclusive right of possession, including all the rights which accompany ownership and is its incident.

Value.—The sum of the desirable qualities which render a thing useful and sought, measured in money; a measure of the relation of two services growing out of the adjustment of mutual needs.

Market Value.—The adjustment of two services, *i. e.,* as between a willing seller and a willing buyer under open conditions of competition.

Cost.—The actual outlay of money, or its equivalent, for a property, structure, or machine.

Past Cost.—The amount of money, or its equivalent, actually expended in the past in creating and building up a property.

Investment.—The amount of capital, or its equivalent, actually expended for a property in the past.

Reproduction Cost.—(Used in this paper generally as a short term for reproduction less depreciation.) An estimate of the cost of recreating a property at the present time under conditions that are humanly possible and practical.

Depreciation.—The lessened value of any property, structure, or machine, due either to its wear, loss of usefulness, growing lack of adaptation, or approaching abandonment.

Appreciation.—The increase in worth of a property, structure, or machine, due to its increasing use, strategic location, the increasing need for its service, or other like influences.

Amortization.—The repayment of an original investment or debt by means of sinking funds, or other moneys set aside from time to time in expectancy of obsolescence.

Public Utility.—A business supplying a public need, and based on a public grant.

Monopoly.—A business having exclusive power of dealing in a service, and thus conducted without competition.

Franchise.—A grant by the public of the necessary rights to do a specific business.

Other definitions might be included, pertaining more to the details of appraisal work, but as it is the intention here to limit the discussion as far as possible to fundamentals, it is believed that the foregoing will suffice.

LAW—THE FOUNDATION.

The fundamental principles of the science of valuation must rest primarily on the law, and to an adequate knowledge of the law must be added a knowledge of the interpretation of the law by the Courts in their decisions and opinions. Courts are not free to reach conclusions on broad general principles; their function is, not to say what the law ought to be, but simply to interpret it as it is. They, as well as economists and engineers, must accept fundamental law as it stands; their duty is to tell us what this fundamental law means.

The function of the lawyer, therefore, in utility valuations, is to call the attention of engineers, economists, and appraisers to the law and to the interpretation of the law by the Courts, and to assist in its application to the facts, and this is a fundamental and important matter. What is to be determined, and the material factors to be taken into account in its determination, are questions of law, exclusively. It is in this connection that the utility commissions, the economist, and the engineer usually make their most serious mistakes. Disregarding the legal basis on which all valuation work must rest, their attempts to decide difficult and important legal questions on the strength of their own general judgment, knowledge, or practice, alone, without any inquiry into the law governing such questions,

naturally adds seriously to the confusion and difficulties surrounding an already complex problem.

The final authority for the interpretation of the law, which is the foundation on which the work of public utility valuation in this country must be built, is the Supreme Court of the United States. Its decisions, until modified, must be regarded as authoritative and final. Next in importance, in interpreting the law, come the decisions and opinions of the higher State Courts, which have been studying and applying the law to questions of this kind for the last fifteen or twenty years, producing an historically connected body of reasoning which no serious student of valuation work can afford to remain in ignorance of or neglect. The utility commissions are all subject to review by the higher Courts, and are, therefore, in effect specialized lower Courts or legalized appraisal boards with enlarged and extended powers.

The interpretation of the law by the Courts has in it little of permanent value, unless it expounds the fundamental law along the lines of sound reasoning and justice. With great responsibilities resting on them, Courts naturally proceed cautiously, rarely stepping outside of the necessities of the particular case in which they have to produce a finding, and yet progressing step by step, so that, as new applications of old principles are needed, there is slowly built up a rational line of reasoning which is of the greatest fundamental value. It must be admitted that Courts oft-times get confused and bewildered when unexpectedly called on to determine purely economic or technical questions. It is not unjust to say that in some cases judges are unfitted by prior training and so handicapped by lack of time as to prevent them from mastering the economic or technical problem before them. Under such circumstances, they sometimes make technical mistakes in their decisions, which are unfortunate because they delay progress. When a Court is bewildered by contradictory expert testimony, it occasionally reveals its embarrassment by criticizing the experts. Courts should realize that truth often lies in a twilight zone, and that this twilight zone may be much narrowed down by the services of good, specialized, and competent witnesses in their calling, and Courts ought to be so skilled in human nature as to be able to disassociate the able and mature from the immature and pretentious witness on specialized subjects.

Taking it all in all, however, Courts are conservative, and their reasoning is usually sound. They are cautious, also, and venture into new and uncharted seas slowly, sounding by sounding. The exceptions prove the rule, and the engineer, in studying their decisions, will not run amiss if he takes the general results of their interpretation of the law as a safe guide. Courts and appraisers are both bound by the law. Whether it is such law as we would make ourselves is outside of the question; the law as we find it must govern us.

The Engineer in Valuation Work.

Although valuation work is based on law, and should adapt itself to the decisions of the Courts, yet the larger portion of the actual work of the valuer will be done in the capacity of the engineer, because almost all utility properties are essentially engineering projects, in which engineering experience, skill, and activity are predominant. The engineer, therefore, building soundly on the fundamental requirements of the law, may proceed to matters more properly within his own jurisdiction, where his accumulated practical experience with the laws of Nature and the constructive works of man will play the larger part.

Economics.

To the resources of the law and to the practical experience of the engineer must be added a third realm of information, which is, in many ways, a distinct domain by itself: that of finance or economics. Too often neither the engineering nor the legal mind is well trained in the great practical world of business, nor realizes the fluctuating value of money, nor is skilled in that class of problems which relates to the conservation of wealth, its limitations, use, and abuse. It is quite true that the more mature engineer should, and often does, know in a general way a good deal about finance, particularly if he has been much occupied with the broader aspects of engineering—that is to say, in those relationships where projects are financed and placed on a sound business basis. The great majority of engineers, however, do not come at all in contact with the preliminary operations under which projects are financed, and are often not well informed, and sometimes regrettably opinionated on matters of this kind. The nature and the duties of members of the legal profession are such that they are probably more often called into consultation as to the larger financial

questions than the great body of engineers, but even the practice of law does not always bring about a practical working knowledge of the ordinary rules that govern and guide the financier.

Utility Experts.

Taking advantage of the fact that there are many fields of knowledge in utility valuation which must be mastered in part, there has sprung into existence of late the term "public utility expert," obviously growing out of the desire to describe a composite calling, able to offer all the knowledge which must be possessed by a public utility valuer in all classes of valuation matters. There can be little question as to the futility of such enterprise.

It is impossible for any man to be a thoroughly competent expert in all phases of even one profession, and the work of valuation requires the co-operation of experts from several professions, and, as well, engineering specialists from each type of utilities to be valued. If there ever was a place where a "Jack of all Trades" is worse than useless, it is in valuation matters. The Courts and commissions are already overburdened by the necessity of listening to ambitious inexperience and voluble plausibility posing as experts, and the all-round expert, who can value anything under the light of the sun, should not be tolerated. If we are to have progress and justice, we should emphasize the importance of having co-operative effort from the most mature class of specialists in every line of specialized endeavor.

Value.

Turning now to some of the fundamental things which the law outlines as a foundation for all valuation processes, we may properly, first of all, inquire what value is. It would appear that this is a very simple question, but it is astonishing how few engineers, for instance, make any distinction at all between "value" and "cost." To grasp clearly the distinction between value and cost is one of the first fundamental principles which an engineering appraiser must learn. Very little reflection is needed to convince one that a thing may be worth much more than it cost, or it may be worth much less. It may even be worth more than it would cost new at the present time, or worth much less. This fundamental conception the legal profession and the economists almost always have well in mind, but it is often a

new thought to the engineer. This is because the engineer is largely dealing with new construction, in which cost most often approximates value.

Let us see what the Courts have said about so simple a thing as value:

VALUE DEFINED.

"Value is the relation of two services. The idea of value entered into the world for the first time when a man said to his brother, 'Do this for me, and I will do that for you.' They had come to an agreement. Then we could say the two services were worth each other." (State *v.* Yates, 10 Ohio, Dec. 150, 158; 19 Wkly. Law Bul., 150.)

"The word 'value' means the exchange power which one commodity or service has in relation to another." (State *v.* Yates, 10 Ohio, Dec. 150, 158; 19 Wkly. Law Bul., 150—citing Walker, "Science of Wealth.")

"The word 'value', in its commonly received signification, means the sum of money a thing will produce to the seller when sold. We are aware that this is not abstractly the true measure of value. The quantity of labor and capital necessary to produce a given article, or, in other words, the actual cost of its production, is the true criterion of its worth. For instance, if a manufacturer be asked the value of a yard of cloth, his opinion of value will be determined by his calculation of the expense of producing a yard of cloth of similar quality. If we ask a retail dealer the value of a yard of cloth, his opinion of its value will be determined by the quantity of money such an article will produce in the market. It is apparent that the marketable value may be affected by a multitude of circumstances which will not in their results extend to the cost of production.

"Value is in its nature so vague and indefinite that no human scrutiny can seize all its constituent parts, and, therefore, opinions of value are admissible in evidence from the necessity of the case." (Town of Rochester *v.* Town of Chester, 3 N. H., 349, 358.)

"The term 'value' in a constitutional provision that property shall be taxed according to value, means the worth of the property as compared with the money of the country—the standard by which all values are regulated." (City of Chattanooga *v.* Nashville, C. and St. L. R. Co., 75 Tenn. (7 Lea), 561, 569.)

"As market value. Bouvier, in his definition of value says: 'This term has two different meanings. It sometimes expresses the utility of an object, and sometimes the power of purchasing other articles with it. The first may be called the "value in use", and the latter the

"value in exchange." Webster recognizes a difference between extrinsic and exchangeable value. But in condemnation proceedings "value" will be held to mean market value.'" (Little Rock Junction Ry. *v.* Woodruff, 5 S. W., 792, 795; 49 Ark., 381; 4 Am. St. Rep., 51.)

"When applied to property, and no qualification is expressed or implied, 'value' means the price which the property could command in the market." (*In re* McGhee's Estate, 74 N. W., 695, 697; 105 Iowa, 9.)

"The word 'value', as defined by Bouvier, has two different meanings. It sometimes expresses the utility of an object, and sometimes the power of purchasing goods with it. The first may be called the 'value in use,' the latter, the 'value in exchange.' For the purpose of the law of eminent domain, the term 'value' has reference to the value in exchange, or market value. There are some cases which seem to hold that the value in use to an owner is to be taken if it exceeds the market value, but it will generally be found, on careful examination, that such cases either refer to the damages accruing to the owner from the taking, and not to the value of the property itself, or they overlook the distinction between the two things. The consensus of the best-considered cases is that for the purpose in hand the value to be taken is the market value." (San Diego Land and Town Co. *v.* Neale, 20 Pac., 372, 374; 78 Cal., 63; 3 L. R. A., 83.)

"The primary meaning of 'value' is worth; and this worth is made up of the useful or estimable qualities of the thing. Ordinarily, when an article of sale is in the market, and has a market value, there is no difference between its value and the market price, and the law adopts the latter as the proper evidence of value. This is not, however, because 'value' and 'price' are really convertible terms, but only because they are ordinarily so in a fair market." (Kountz *v.* Kirkpatrick, 72 Pa. (22 P. F. Smith), 376, 386; 13 Am. Rep., 687.)

"Value of Railroad. The value of a railroad track at any given time is not the original cost nor its ultimate cost after years of expenditure in repairs and improvements. On the other hand, its value cannot be determined by ascertaining the value of the land included in the roadway, assessed at the market price of adjacent lands, and adding the value of the cross-ties, rails, and spikes. The value of land depends largely upon the use to which it can be put, and the character of improvements upon it. The assessable value for taxation of a railroad track can only be determined by looking at the elements on which the financial condition of the company depends;—its traffic as evidenced by the rolling stock and gross earnings in connection with its capital stock." (Pittsburg, C. C. and St. L. Ry. Co. *v.* Backus, 14 Sup. Ct., 1114, 1118; 154 U. S., 421; 38 L. Ed., 1031.)

"The value of a railroad for purposes of taxation is not the mere value of its right of way, roadbed, its structure, its depot ground and structures thereon, considered by themselves, but the value of all these as an operating going concern; this value being in general determinable by the profits which result from its operation." (State *v.* Austin and N. W. R. Co., 62 S. W., 1050; 94 Tex., 530.)

It must be fully realized that, in public utility valuation, we are endeavoring to ascertain the fair value as between the public, on the one hand, with its needs for the service, and the owners of the property which has been devoted to the public use, and is practically incapable of extricating itself from that service, and, therefore, peculiarly dependent on just and fair treatment for its conservation.

Property.

Another of the fundamental concepts which should be well established in the minds of utility commissions and engineers is the meaning of the word "property." Here, again, it seems strange that any considerable difference of opinion should arise about a definition which has been so long a subject of inquiry as that of the word "property," and yet lack of a clear idea as to what is being appraised in a public utility is mainly responsible to-day for the wide divergence of views as to how to proceed and what to value.

One might well ask how it is possible to accomplish anything without preliminary agreement as to so fundamental a proposition. It must be conceded that it is fundamental, in valuation work, to know what it is we are to value, yet engineers and appraisers generally will pass out opinions as to the value of plants without any intelligent inquiry as to the fundamental conception of what they are valuing. This is where the law and the Courts lay a foundation that ought to be clearly in mind before one tries to discuss details or proceed to estimate.

There arise, from the prevailing fundamental misconceptions, two opposing schools of thought, born on opposite sides at interest, which, being radically at variance with each other, can, of course, find no common ground for agreement in detailed methods.

On one side is ranged a school of thought, consisting, for the most part, of the majority of those advocates of municipal ownership and operation who have first approached the subject largely from that side of the question. This school of thought, as well as

certain of the more recently formed utility commissions and their staffs, who have been influenced by such arguments, conceive that the utility should be examined largely from the standpoint of the actual cost, or treated as a past investment, and that it can be considered to have value only in so far as actual cash or money has been put into the plant and property. The common viewpoint of this class of appraisers is that this past cost should be stripped of all accretions and increments of growth that are not represented at some time or other by actual expenditures. Certain extremists of this class of thinkers would even strip the actual "past investment" of its appreciation in the value of land and other like appreciations, while confining value to the actual past cash cost. They would further deduct from it any depreciation that might be found to have accrued.

On the other hand, as an opposing school of thought, there has developed a class of valuers who hold that the plant and business of a utility company together constitute a "property", which may have accretions in value outside of actual cash investment, and that such property may appreciate through such accretions, just as it may depreciate through age, lack of adaptability, or other causes.

Now this is a very fundamental and important difference of opinion, and needs intelligent and fair-minded investigation before we are at all competent to enter into detailed questions of valuation. Are we to find the actual dollars that have been invested in a plant and business, or are we to find the value of the resulting "property"? What does it profit us to discuss details when this serious and fundamental difference exists? The two schools of thought that have approached this subject of valuation differ so widely on this question that it is safe to say that much more than one-half of all our time in Courts and before commissions is taken up with testimony and argument directed to the one or the other of these two theories. Most of the recent literature on the subject of valuation work bears evidence of a fatal disagreement on this one fundamental point.

Curiously enough, this important question is emphatically settled by the highest authority we have, or can hope to have: the Constitution of our country as interpreted by the Supreme Court of the United States.

Property is a well-defined word in the meaning of the law, and the Fourteenth Amendment to the Constitution of the United States

is fundamentally the authority under which the Courts proceed in the determination of fair value of property for use in the fixing of rates or the ascertainment of value for condemnation or sale. It reads, in part, as follows:

"Nor shall any State deprive any person of life, liberty, or property without due process of law."

The provision of the Fifth Amendment to the Constitution, controlling condemnation cases by the United States, reads as follows:

"Nor shall private property be taken for public use without just compensation."

In the Constitutions of the States are similar provisions in which the word "property" is used. The significant thing is, that whether we look to the Fourteenth or to the Fifth Amendment, or to the State Constitutions, what shall not be taken is "property." The Fifth Amendment prohibits the United States from taking property without compensation. The Fourteenth Amendment prohibits any State from taking property without due process of law.

Nor has there been left undecided the question, whether property devoted to the public use shall be treated differently in respect to its ownership and right of protection from other private property, as will be shown by later quotations. What is here to be emphasized is that the fundamental law of our land does not use the terms "investment", or "cash cost", or "past cost", or "actual expenditure", or any one of the numberless approaches to actual cash investment that would be used if a property were not precisely meant, with all that the term involves.

However, it is now proper to inquire further: What is property? Let us look for a moment at some of the more common legal definitions.

PROPERTY DEFINED.

"The sole and despotic dominion which one man claims and exercises over the external things of the world in total exclusion of the right of any other individual in the universe." (2 Bla. Com., 2.) "The right to possess, use, enjoy, and dispose of a thing (56 N. Y., 268) which is in itself valuable (4 McLean, 603). The right or interest which one has in lands or chattels. (6 Binn., 94; 4 Pet., 511.) The free use and enjoyment by a person of all his acquisition without any control or diminution, save only by the law of the land. (2 Ark., 291; 61 Hun, 571.) The right of a person over a thing (*in rem*) indefinite in point of user." (Austin's Lectures.)

"Webster defines 'property' to be the exclusive right of possessing, enjoying, and disposing of a thing; ownership; an estate, whether in lands, goods or money." (Spring Valley Water-works *v.* Schottler, 62 Cal., 69, 72, 84. See also Banning *v.* Sibley, 3 Minn., 389, 404 (Gil., 282); McKeon *v.* Bisby, 9 Cal., 137, 142; 70 Am. Dec., 642.)

"The term 'property', as used in the Constitution, is not confined to tangible objects which can be passed from hand to hand, but includes those rights of possession, disposal, management, and of contracting with reference thereto, which render property useful, valuable, and a source of happiness, the right to the pursuit of which is preserved." (State *v.* Kreutzberg, 90 N. W., 1098, 1100; 114 Wis., 530; 58 L. R. A., 748; 91 Am. St. Rep., 934.)

"The term 'property', as used in the Constitution, Articles 1 and 17, forbidding the taking of property for public use without adequate compensation being made therefor, means not only the thing owned, but also every right which accompanies ownership and is its incident." (Gulf, C. and S. F. Ry. Co. *v.* Fuller, 63 Tex., 467, 469; Ft. Worth and R. G. Ry. Co. *v.* Jennings, 13 S. W., 270; 76 Tex., 373; 8 L. R. A., 180.)

"Anticipated Profits. Anticipated profits are not, and cannot be, held and regarded as property in the ownership or possession of him who owns the article out of which profits are expected to flow. The property is one thing, and remains untouched. The profits are *in esse*, and cannot be claimed as property." (Munn *v.* People, 69 Ill., 80, 91.)

"The term 'property', in its broad sense, includes even a franchise, so the word 'property' as used in the statute providing for the taxation of all property not specifically exempted, includes a franchise, and hence a tax on a turnpike is property." (Frankfort, L. and N. Turnpike Road Co. *v.* Commonwealth, 6 Ky. Law Rep., 391, 392.)

"Property in its broadest and most comprehensive sense, includes all rights and interest in real and personal property, and in easements, franchises, and in corporeal hereditaments. That which may be taken for public uses is not exclusively tangible property. Wherever the right of eminent domain exists, whatever exists in any form, whether tangible or intangible, may be subjected to the exercise of this power, and may be appropriated to public use when necessity demands. A franchise is nevertheless a right in the nature of 'property' within the sense that the word is used in the eminent domain law, and no reason is perceived why it could not be condemned for the use of the public. We know of no policy of the law that would forbid the taking of such property for public uses." (Metropolitan City R. Co. *v.* Chicago W. D. Ry. Co., 87 Ill., 317, 324.)

"Water. Water, when reduced to possession, is property, and it may be bought and sold, and have a market value; but it must be in actual possession, subject to control and management. Running water in a natural stream is not property, and never was. The construction of a dam across an outlet of a lake or a river or a creek for the purpose of securing power with which to operate mills or factories is not a reducing of the water to possession, or to control or management, in such a sense as to change its legal character and make it property. If it did, the owner at times might find it difficult to control, especially in case of freshets or floods. He might have trouble in preventing it from being precipitated upon the lands of his neighbors below, and find it unpleasant to respond to them for the damages caused by his property." (City of Syracuse *v.* Stacey, 62 N. E., 354, 355; 169 N. Y., 231.)

It seems to be apparent from these definitions that a property, even in a public utility, is not a mere aggregate of separate structural materials, priced as would be the stock of a shopkeeper, nor can such property be properly confined to a past investment which must be traced out, dollar by dollar, to show how much actual cash has really been put into it. A property, even in a public utility, is a right of possession in a complete entity actively engaged as a co-ordinate whole. This property, as a whole, lives through the ordinary vicissitudes of life, and may have its accretions of value, due to the growth, expansion, and enterprise of the adjacent population, as well as its depreciations, due, among other things, to lessening demand, structure disuse, or changes in the arts. These may be outlined generally, as follows:

ORIGINAL COST.

Depreciations.		Appreciations.
(1) Wear and tear on machinery and plant. Decay.		(1) Growth of population and demand on original installation.
(2) Decreased usefulness of plant due to shifting population.		(2) Enlargements and extensions.
(3) Changes in general demand producing obsolescence.		(3) Growth of population on extensions.
(4) Improvements in the art, requiring replacement.		(4) Growth of habit, increasing use of service per capita.
(5) Changes required in the source or method of manufacture.	Period of growth and usefulness, or of decline and abandonment.	(5) Increasing cost of replacement.
(6) Insufficiency of original installation by reason of increased population.		(6) Increasing value of real estate.
(7) Checks to growth of population or surrounding industries.		(7) Increasing cost of materials and labor.
(8) Decrease in price of labor and materials.		(8) Increased business due to enterprising management.
(9) Increased competition.		(9) Development of new uses for supply or service.
(10) Imprudent expenditures.		(10) Economies in production effected by good management.

PRESENT VALUE.

A property, therefore, is like an individual. It is born; it goes through its youthful training; it encounters its vicissitudes; it comes to its maturity, or period of usefulness; it expands with the community perhaps; it has its period of decline, and is supplanted by better methods of service or production; eventually it will die, at least so history teaches us. Constantly, however, we find valuers (usually representing what may be called the buyer side of the question) who, through mistaken sympathy, leading to unconscious unfairness, or through ignorance of all sides of the question, or more rarely, it must be regretfully said, through direct prejudice or deliberate intent, would deprive a utility property of its natural growth in value, on the theory that they are dealing here with something other than private property, devoted to the public use. In other words, they would deal with private property devoted to public use, as property the natural growth of which could be checked, or portions of the value of which could be appropriated to the public ownership because of its public use.

Of course, it would be a very interesting and perhaps even profitable inquiry to discuss the question as to whether the Constitution of the United States, as construed by the higher Courts, may be wrong in its theory of property. It might also be of great interest and profit to inquire whether interest is moral, and to raise many other fundamental questions of a like nature. Many intelligent people can be found (and some of them are in valuation work) who would like to see the Constitution revised, at least so far as relates to private property devoted to public use. Such people would not dream of appropriating private property for private use, but are apparently quite willing to appropriate, in part at least, private property devoted to the public use.

The right to hold property, with all its increment of growth, is fundamental to our civilization. Without this right our civilization would vanish. It is to the public interest that this should continue to be so, for the moment we begin to treat private property, devoted to the public use, as public property, then private property will cease to be available for the public uses and the needs of the people, to the detriment of every one. What we are properly striving for, in all the recent movement toward regulation of private property devoted to public use, is not the lessening of the value of that property, but the

correction of abuses and evils which grow out of its improper use or abuse. Wherever, then, we find wide differences of opinion as to the value of a property, devoted to the public use, we will usually find that a large part of the difference is due to differing conceptions as to what it is that is being valued, and differing definitions resulting from that fundamental departure.

If appraisers, and especially engineers, could once get it thoroughly understood that it is not their function to say what is to be appraised—that function belongs to the law, and such points are fundamentally determined by the Constitution of the United States, as interpreted by the Supreme Court—then most of our troubles and confusion in appraisal work would be over.

The Right of Regulation.

The leading case in the Supreme Court of the United States upholding the power to regulate the rates of public utilities was handed down as far back as 1876, and is based on the proposition that one who devotes his property to a public use:

"In effect grants to the public an interest in that use, and must submit to be controlled by the public for the common good to the extent of the interest he had thus created.

"He may withdraw his grant by discontinuing the use, but so long as he maintains the use he must submit to its control." (Munn *vs.* Illinois, 94 U. S., 43, 24th Law Ed.)

It is interesting to examine historically the widening acceptance which has been given to this principle, even by the Courts themselves. Little by little it has been extended from pure monopolies into the domain of quasi-monopolies, and finally to public utilities that are competitive, such as lighting, heat, and power; so that now it is virtually conceded that all business in which public grants are necessary is subject to public regulation. It is now coming to be well understood that a successful public utility property, to give good service, should, as a rule, be a monopoly. The old idea which prevailed a generation ago, and was recognized by the laws of such States as California—that it was a good thing to have rivalry in public service corporations—has passed, and we recognize that a regulated monopoly is, all things considered, by far the best method of conserving our resources and serving ourselves.

When, however, we come to value a monopoly as a property, and especially where a monopoly is subjected to rate regulation, we appear to come to a difficulty that its value may depend, to a large extent, on the rates, and as the proper rates are obviously dependent in part on the value, we seem at first glance to be reasoning in a circle. This difficulty, however, is not as great as it appears, because on closer investigation we shall find that, when a property consists largely of a physical plant, we can find a proper value for the property, even though its value depends in part on a regulated rate of return. This is also possible where the rates must afford a fair return, which rate of return, however, must be assumed in advance. In other words, where there is a fixed physical plant and a fixed fair rate of return, a definite value results.

In valuing a regulated monopoly or partial monopoly, we cannot, of course, proceed as we would with an ordinary business, fluctuating with its good-will, nor can we properly capitalize existing income independent of physical plant. We are, therefore, evidently limited to methods in which value is not wholly dependent on income, because the income is to be regulated. We are usually obliged, therefore, to reason from the basis of the physical property, plus the cost of financing it into proper and remunerative income.

A great deal of exception has been taken to this embarrassing condition, because it at once does away with a most common and natural method of arriving at value, that of capitalizing income. It is difficult, however, to see how this can be overcome, so long as rate regulation is a fundamental precedent to a valuation.

The Supreme Court of Wisconsin, in the recent Appleton Case (142 Northwestern Reporter, 476, May 31st, 1913), evidently feeling the embarrassment of the situation, says:

"The commercial value of the business in full operation and entitled to charge reasonable rates for its service must, however, be considered as approximating the compensation which should be allowed for the property; in other words, the sum which the business should be capitalized for, in order that the owner should receive a reasonable return on the investment when the business is conducted with reasonable business skill, and charges such reasonable rates for service as the law permits."

This rule would bring permissible net income, or, as the English authorities put it, "maintained and maintainable net income," into a

position of prominence at once, were it possible to define maintained and maintainable income, or, "such reasonable rates for service as the law permits," entirely aside from all consideration of the property devoted to the public use itself, but the Wisconsin Supreme Court nowhere else in its opinion gives us any clue as to how this "reasonable rate" is to be defined, and ends by approving the findings of the Commission, based on examination of the physical property in connection with other evidences of value.

There seems to be no escape from the conclusion that, so long as rates of return are to be regulated, they cannot be used as a sole foundation for ascertaining value. Some other and more devious path is forced on our attention, in which the importance of the return is, if not altogether eliminated, at least so minimized as to lose its preponderating influence.

THE STARTING POINT.

An examination of their decisions will show that the Courts have reached a definite conclusion that the proper interpretation of the Constitution requires them to insist on two essentials:

(*a*) That what is to be valued is the "property."

(*b*) That what is to be ascertained is the present value of this property.

This has compelled the Courts to require such an investigation as will present to them all the factors necessary for the determination of the present value of the property devoted to the public use.

The following is an extract from Minnesota Rate Cases, 230 U. S., 352, 434:

"(1). The basis of calculation is the 'fair value of the property' used for the convenience of the public. Smyth *v.* Ames, *supra* (p. 546). Or, as it was put in San Diego Land and Town Co. *v.* National City, *supra* (p. 757), 'What the company is entitled to demand, in order that it may have just compensation, is a fair return upon the reasonable value of the property at the time it is being used for the public.' See also San Diego Land and Town Co. *v.* Jasper, *supra;* Willcox *v.* Consolidated Gas Co., *supra.*

"(2). The ascertainment of that value is not controlled by artificial rules. It is not a matter of formulas, but there must be a reasonable judgment having its basis in a proper consideration of all relevant facts. The scope of the inquiry was thus broadly described in Smyth *v.* Ames, *supra* (pp. 546-547): 'In order to ascertain that value, the

original cost of construction, the amount expended in permanent improvements, the amount and market value of its bonds and stock, the present as compared with the original cost of construction, the probable earning capacity of the property under particular rates prescribed by statute, and the sum required to meet operating expenses, are all matters for consideration, and are to be given such weight as may be just and right in each case. We do not say that there may not be other matters to be regarded in estimating the value of the property. What the company is entitled to ask is a fair return upon the value of that which it employs for the public convenience. On the other hand, what the public is entitled to demand is, that no more be exacted from it for the use of a public highway than the services rendered by it are reasonably worth.'"

Ames *vs.* U. P. Railway, 64 Fed., 165.

San Diego Land and Town Co. *vs.* National City, 74 Fed., 74 (189) (Ross): "In my judgment it is the actual value of the property at the time the rates are to be fixed that should form the basis upon which to compute just rates, having at the same time due regard to the rights of the public and to the cost of the maintenance of the plant and its depreciation by wear and tear."

Cotting *vs.* K. C. Stock Yards (1897), 82 Fed., 850, 854. "If improvements have been made in the vicinity of the property, the growth of the city or town where it is located, the building of railroads, the development of the surrounding country, and other like causes give property an increased value, the owner cannot be deprived of such increase by legislative action which prevents him from realizing an income commensurate with the enhanced value of his property."

Smyth *vs.* Ames, 1898, 169 U. S., 466; 18 Sup. Ct., 418 (Justice Harlan). "We hold, however, that the basis of all calculations as to the reasonableness of rates * * * must be the fair value of the property being used by it for the convenience of the public."

Willcox *vs.* Consolidated Gas Co., 212 U. S., 19-52 (1909); 29 Sup. Ct., 192-53.

This last case is an interesting one, as it applies this line of reasoning to some very extreme conditions, and it is a recent case. It may be well to quote from the Circuit Court (157 Fed., 849), from which the appeal was taken, more fully, as follows:

"It is impossible to observe this continued use of the present tense in these decisions of the highest court without feeling that the actual or reproductive value at the time of inquiry is the first and most important figure to be ascertained, and these views are amplified by San Diego Land Co. *v.* Jasper (C. C.), 110 Fed., at page 714, and Cotting *v.* Kansas City Stock Yards (C. C.), 82 Fed., at page 854.

where the subject is more fully discussed. Upon reason, it seems clear that in solving this equation the plus and minus quantities should be equally considered, and appreciation and depreciation treated alike. Nor can I conceive of a case to which this procedure is more appropriate than the one at bar. The complainant by itself and some of its constituent companies has been continuously engaged in the gas business since 1823. A part of the land in question has been employed in that business for more than two generations, during which time the value of land upon Manhattan Island has increased even more rapidly than its population. So likewise the construction expense, not only of buildings, but of pipe systems under streets now consisting of continuous sheets of asphalt over granite, has enormously advanced.

"The value of the investment of any manufacturer in plant, factory, or goods, or all three, is what his possessions would sell for upon a fair transfer from a willing vendor to a willing buyer, and it can make no difference that such value is affected by the efforts of himself or others, by whim or fashion, or (what is really the same thing) by the advance of land values in the opinion of the buying public. It is equally immaterial that such value is affected by difficulties of reproduction. If it be true that a pipe line under the New York of 1907 is worth more than was a pipe line under the city of 1827, then the owner thereof owns that value, and that such advance arose wholly or partly from difficulties of duplication created by the city itself is a matter of no moment. Indeed, the causes of either appreciation or depreciation are alike unimportant, if the fact of value be conceded or proved; but that ultimate inquiry is oftentimes so difficult that original cost and reason for changes in value become legitimate subjects of investigation, as checks upon expert estimates or bookkeeping inaccurate and perhaps intentionally misleading. *Cf.* Ames *v.* Union Pacific R. R. (C. C.), 64 Fed., at pages 178, 179. If 50 years ago, by the payment of certain money, one acquired a factory and the land appurtenant thereto, and continues to-day his original business therein, his investment is the factory and the land, not the money originally paid; and unless his business shows a return equivalent to what land and building, or land alone, would give if devoted to other purposes (having due regard to cost of change), that man is engaged in a losing venture, and is not receiving a fair return from his investment, *i. e.*, the land and building. The so-called 'money value' of real or personal property is but a conveniently short method of expressing present potential usefulness, and 'investment' becomes meaningless if construed to mean what the thing invested in cost generations ago. Property, whether real or personal, is only valuable when useful. Its usefulness commonly depends on the business purposes to which it is or may be applied. Such business is a living thing, and may flourish

or wither, appreciate or depreciate; but, whatever happens, its present usefulness, expressed in financial terms, must be its value.

"As applied to a private merchant or manufacturer, the foregoing would seem elementary; but some difference is alleged to exist where the manufacturer transacts his business only by governmental license —whether called a franchise or by another name. A license, however, cannot change an economic law, unless a different rule be prescribed by the terms of the license, which is sometimes done. No such unusual condition exists here, and, in the absence thereof, it is not to be inferred that any American government intended, when granting a franchise, not only to regulate the business transacted thereunder, and reasonably to limit the profits thereof, but to prevent the valuation of purely private property in the ordinary economic manner, and the property now under consideration is as much the private property of this complainant as are the belongings of any private citizen. Nor can it be inferred that such government intended to deny the application of economic laws to valuation of increments earned or unearned, while insisting upon the usual results thereof in the case of equally unearned, and possibly unmerited, depreciation.

"I think the method of valuation applied by the report to land, plant, mains, services, and meters lawful."

On appeal to the United States Supreme Court, the above position of District Judge Hough was approved. Justice Peckham, in delivering the opinion of the Court, says (212 U. S., 19, 52; 29 Sup. Ct., 192; 53 L. ed., 382, January 4th, 1909):

"And we concur with the court below in holding that the value of the property is to be determined as of the time when the inquiry is made regarding the rates. If the property, which legally enters into the consideration of the question of rates, has increased in value since it was acquired, the company is entitled to the benefit of such increase. This is, at any rate, the general rule. We do not say there may not possibly be an exception to it, where the property may have increased so enormously in value as to render a rate permitting a reasonable return upon such increased value unjust to the public. How such facts should be treated is not a question now before us, as this case does not present it. We refer to the matter only for the purpose of stating that the decision herein does not prevent an inquiry into the question when, if ever, it should be necessarily presented."

It is apparent, from these opinions, that there is no escape from the conclusion that in all ordinary cases we must value a public utility as a "property" and find its value as of to-day. Nothing short of a reversal of its own opinion by the Supreme Court, a new Consti-

tution for the United States, a revolution, or what would be (in the writer's opinion at least) infinitely worse, the recall of judicial opinion, could alter this present status.

If, therefore, we must value a utility as a property, and value it now, it is quite obvious why the Courts have concluded that reproduction cost, less depreciation, of the existing property (or, as we will call it for the sake of brevity, "reproduction") is usually a leading and most important line of evidence to aid in arriving at value. The Courts do not suggest the neglect of any other lines of evidence, but, under normal circumstances, they place weight on such evidence as tends to show the value of the property now.

The general trend of recent decisions has been to make reproduction cost methods a controlling and important line of evidence as to value under normal conditions.

In a Minnesota rate case—Steenerson *vs.* Great Northern Railway, 69 Minn., 353, October 20th, 1897—the Court said:

"Then the burden is on the railroad company to show that the rates fixed by the Commission are unreasonable, and for this purpose the original cost of the road, the amount of its present fixed charges, and its history, are material only so far as they show what it would now cost to reproduce the railroad."

In 1911, the later Minnesota rate cases, Shepard *vs.* Northern Pacific Railway Co., 184 Fed., 765, April 8th, 1911:

"The master rightly decided that the cost of reproducing this property new was a more rational and reliable measure of its real value than the original cost of its acquisition and construction or the market values of the stocks and bonds of the companies, and upon that basis he made his findings."

In San Diego Land and Township Co. *vs.* National City, 74 Fed., 79, May 4th, 1896, the Court says it is "Present value", and, on appeal, Justice Harlan refers to "reasonable value of the property at the time it is being used for the public." (174 U. S., 739.)

In the Consolidated Gas Case *vs.* the City of New York, 157 Fed., 849, Judge Hough said:

"In every instance, however, the value assigned in the report is what it would cost presently to reproduce each item of property in its present condition, and capable of giving service neither better nor worse than it now does. As to all the items enumerated, therefore, from real estate to meters, inclusive, the complainant demands a fair

return upon the reproductive value thereof, which is the same thing as the present value, properly considered. To vary the statement: complainant's arrangements for manufacturing and distributing gas are reported to be worth the amounts above tabulated if disposed of (in commercial parlance) 'as they are.'

"Upon authority I consider this method of valuation correct. What the court should ascertain is 'fair value' of the property being used (Smyth *vs.* Ames, 169 U. S., 546); the 'present' as compared with 'original' cost; what complainant 'employs for the public convenience' (169 U. S., at page 547); and it is also the value of the property at the time it is being used (San Diego Land Co. *vs.* National City, 174)."

The Courts are extremely cautious not to commit themselves to reproduction methods, as a rule, for finding value, and this is because they see clearly that exceptional conditions might make the evidence of reproduction methods at times misleading and dangerous. As a matter of practical inquiry, however, under the conditions as ordinarily met with, and in the great majority of valuation cases, the Courts pay special attention to reproduction methods, and the largest portion of the testimony in such cases is usually directed to this line of evidence, for the very good reason that, unless abnormal conditions are present, it is the method which will most nearly disclose the value of the property as of to-day.

Let us inquire further what general procedure we may adopt in valuation work to be in harmony with the reasoning of the Courts on this subject.

PRACTICAL METHODS OF UTILITY VALUATION.

The function of valuing a public utility property consists essentially of two distinct operations:

First.—The collection of all the facts and evidence tending to throw light on the question of value; and

Second.—The judicial function of weighing the evidence, apportioning to each line of evidence its relative degree of importance, and reasoning from this summation to the broader question of value.

In valuation work, much confusion of thought arises from the very human anxiety to reason on the broader question of value before all the data are accumulated, properly digested, and placed in their proper perspective.

The facts and evidence desirable for a full understanding of the value of a utility property are generally as follows:

First.—The cost to replace the property at the present time as a going concern.

This line of investigation requires:

(*A*) An inventory of the present existing physical property, used and useful for the purpose of the utility.

(*B*) A statement of its earnings and expenses for a preceding series of years, not less than five, and preferably ten or more.

(*C*) Computation of the cost to replace the property now, and reproduce its income as a going concern, all in a way which is humanly possible.

(*D*) An estimate of the loss of value in the existing property, by reason of age, changing requirements, advances in the arts, or use.

Second.—The market value of the securities of a property, where a broad and representative market for such securities can be said to exist.

Third.—The value of other properties similarly situated, where such value has been carefully and fully determined. This can be used only as a general check, however.

Fourth.—The conditions which surround the particular property to be valued, such as the character of population, its past growth and present business enterprise; the presence or absence of judicious management of the utility property, and its conservative extension; freedom of the property from mishaps; the value of the service to the individual consumer; competition, if any, and possible future contingencies arising both from changes in public demand or substitution of other service more acceptable to the population; and, finally, the legal status of the property, and attitude of the public and its representatives toward it.

Fifth.—The Courts hold that, where past cost can be obtained, it is competent evidence. The fact that it is competent does not mean that it is controlling, and does not indicate the weight it should have. What weight it should have in

any case depends on whether or not the conditions when the past cost was incurred are the same as the conditions at the date of the valuation. If they are, it is valuable evidence. In proportion as the conditions differ, the weight to which this testimony is entitled grows less. It is for this reason that recent costs, which presuppose unchanged conditions, are treated as valuable factors, while ancient costs are usually valueless.

The judicial reasoning which should follow the compilation of these lines of inquiry is as follows:

First.—To be assured that the evidence is without material error.

Second.—To determine the amount of credence to give to the witnesses who introduce evidence. This may be based largely on their practical experience in the matters in hand, as well as their conscientiousness and freedom from bias.

Third.—To determine that each independent line of evidence introduced is in itself logical and complete, and is not confused with any other line of evidence, such as the mixing up of past cost with present cost to reproduce, etc.

Fourth.—To decide what weight should be given to the various lines of evidence introduced.

Fifth.—To determine in each case the extent to which each line of evidence is a material factor in measuring value, and if it is not such a factor, to reason why not.

Sixth.—To determine in particular whether the cost of reproduction, including the cost of reproducing the present income of the company, is or is not the most important factor in reaching a sound conclusion as to value.

Seventh.—To conclude finally on the value, from a broad and adequate review of all the facts, with an open mind, in a judicial manner, and as a matter of common sense.

Eighth.—To check the value so obtained by such lines of inquiry as will properly be of aid.

In weighing evidence as to value, it is important to remember:

First.—That value is, after all, a matter of intelligent opinion, based on all the ascertainable facts with special reference to the conditions of the case in hand.

Second.—That cost is not usually coincident with value.

Third.—That the Courts uniformly have insisted that it is the present value, or value now, that must control.

Fourth.—That reproduction is usually the most direct line of evidence bearing on value now.

Fifth.—That cost now is not necessarily value now, for it is easily conceived that, under many circumstances, a plant may be worth more or less than even the cost to reproduce it now.

Sixth.—That the weight to be given the evidence of individual valuers is in direct proportion:

(*a*) To the extent of their direct past practical experience;

(*b*) To their breadth of view and study in callings other than their own profession affecting valuation work;

(*c*) To their character and standing as fair and unbiased men, used to considering both sides of the questions at issue in an impartial manner.

Reproduction.

The foregoing discussion, quoted opinions, and outline of valuation procedure, demonstrate why reproduction has assumed its importance in public utility valuation. It has not been originated by engineers to torment the Courts, but it has been concluded by the reasoning of the Courts to be a logical and important line of evidence, and the Courts have compelled the Engineering Profession to work out its details in a practical way, and in a manner that is humanly possible.

In considering reproduction, however, we are confronted, first of all, by the fact that the more fully it is examined the more it is seen to be a hypothetical proceeding, using this term in its legal sense. Nothing about it is so real as the results to the owner. The more it is studied the more it must be conceded that reproduction is purely a conceptual forecast in a most difficult and specialized art—that of financing and building especially designed public works for distinct kinds of public service. Even the art of constructing public service plants is comparatively new; the number of engineers specially skilled in the broader aspects of such arts is relatively few; and the mature and experienced engineers in such specialties are fewer still. It is quite obvious that in the immature thought of impractical and in-

experienced minds, reproduction can be made absurd to the point of ridicule, and, even in the hands of an experienced, painstaking, and unbiased investigator, it is questionable whether the full, proper, and natural sequence of events necessary to the hypothetical rebuilding of visible property can always be properly and intelligently thought out with any reasonable degree of thoroughness and proper prophecy, without omission of matters which would probably unexpectedly occur in any actual procession of events.

Conceptual Processes.

Let us see what it is that the Courts have asked us to do when we are made to undertake the task of reproducing. Having familiarized ourselves with the existing property which is to be valued, we must, in imagination, as a prolonged conceptual process, retrace, step by step, every feature of the procedure that would have to be taken if that property were completely wiped out, and we were responsible for the task of entirely rebuilding it in a reasonable length of time and in a manner which is humanly possible. Should we unconsciously omit one important step necessary to that rebuilding, we will fail to reach the full cost. Being in possession of retrospect, we are constantly subject to the temptation to substitute an easy and virtuous hindsight for that real lack of foresight which would be our condition in actual life and under ordinary conditions. If we do full justice to the procedure, we must trace two lines of human endeavor, that is, the mental labor as well as the physical labor which produced the result before us. Mental endeavor will include the development of the project and the preliminary steps of promotion. This must include the necessary time and pains that would naturally be taken to see that the project is practical, the time and attention necessary to negotiate the franchise, investigate the methods of service, and estimate the preliminary work. We must at every step of the way outline to ourselves the necessary lengths of interlapsing time to do all these things, as well as the amount of labor to design, contract for, and actually build each structure. Much of our cost will be entirely dependent on the length of time we assume for these practical operations. It is quite certain that the most trained imagination, practically acquainted with similar construction, will not always entirely succeed in redeveloping every detail involving expense

in such a procedure. The Courts, in advocating reproduction as a valuable line of inquiry, have probably thought that a hypothetical reconstruction was simple. At first glance it would appear simple; so many pounds of rail at so much, so many ties at so much, and total so much. Any one coming to the task of reproducing a property in this spirit has singularly misconceived the extent and complexity of the problem before him.

THE SIDEWALK ILLUSTRATION.

The fallacious attitude of many estimators can best be shown by a simple illustration of sidewalk construction. You sit in your library window with your little boy and watch the construction of a sidewalk across the street, and you ask your boy how sidewalks are built. In all probability he will reply, with his eye on the scene, "Why, they take crushed stone and cinders and sand and cement and mix them up and spread it out to dry between planks that keep the edges straight; then they tamp it all down, smooth it over, leave it to dry, and when it is done they clean up and go away."

Now, this is really about all the average grown person sees of engineering work. Even some judges and economists would hardly give a fuller answer. They might add that the sidewalk costs a certain sum per square foot, because the sidewalk contractor received that sum, and that estimate of cost would be the general impression of a good many mature people as to the cost of sidewalks.

Let us analyze the matter a little further. Assume that you are familiar with municipal engineering, and you will find in your mind's eye vastly more things that have to be done than your boy could see. A petition from the property owners has to be presented to the common council of the city, with the signatures of the majority of the frontage owners attached thereto. The council perhaps sends an inspector, who reports on the matter. A committee of the council deliberates and reports back favorably to the council. An ordinance has to be drawn by the law department of the city, the engineering department submits an estimate for the cost of the work, and the council then passes the ordinance. A warrant is issued from the proper court for the spreading of the assessment and commissioners are appointed who apportion the tax and make their report. A hearing is held at which objectors may appear. The assessment is perhaps

finally confirmed, and a warrant issued to the county or city collector, the tax bills are sent out, and the moneys are received by the treasurer.

In the meantime, the city engineering department has drawn up specifications for the work, describing minutely just what is to be done, the kind of material to be used, and its method of mixing and laying. Notices are inserted in the newspapers calling for bids. Contractors make their estimates and appear at the letting; the bids are received, opened, and tabulated; a report is made by the engineer to the council; and the work is thereupon let to the lowest bidder. A contract is drawn up by the law department, bonds of a proper amount are deposited, and the contract is signed. In the meanwhile the city engineer sends an assistant who stakes out the established grade which had heretofore been fixed by the council from surveys of the city by the engineering department. An inspector is appointed to see that the specifications and contract are complied with. In the meantime the contractor had figured out the necessary quantities of material, purchased the proper quality, and had hauled it to the work. He then engaged a mason and helpers, a foreman and laborers, and then—your little boy saw them appear on the street to do the work.

Now, what was the real cost of that sidewalk? Certainly the city treasurer's books do not show it. Did the engineer in charge know? Did the contractor know? Did your little boy know? Did any one ever really know?

Public Misconceptions.

So it is with the average public utility valuation. The public thinks of it as a list of visible material which must simply be priced, and when the engineer talks about preliminary expense, length of time, methods of construction, plant financing, overhead charges, developing the business, and the like, the public thinks he is trying to inflate the valuation by what it likes to call "intangible" items.

The fact is, as has recently been well emphasized by M. E. Cooley, M. Am. Soc. C. E., in an address before the Annual Meeting of the Western Society of Engineers,* that neither the Courts nor the public have any clear idea of the real cost of building up a property, and it is a public duty for the Engineering Profession to point out all the necessary elements of cost which would be incurred in a real enter-

* *Journal*, Western Soc. of Engrs., January, 1914, p. 1.

prise, and to insist that hypothetical reproduction estimates must be made in a manner which is humanly possible and practically feasible, such as would be the case in real life and in line with real practical experience.

Most attempts at hypothetical reproduction produce estimates that arrive at a less cost of reproduction than would be found necessary in real actual building, by reason of the before mentioned tendency to omit minor items, overlook necessary auxiliaries, or fail to appreciate all the contingencies that are likely to occur in an actual undertaking. All engineers realize that the less we know about a given line of building work, the more apt we are to underestimate it, and almost every one who has built a house can recall how much they underestimated the final cost, and why.

Reproducing the Income.

When we reproduce a property, we must reproduce all of the property there is; that is, we must reproduce it up to its present state of efficiency, including the cost of financing the plant to the point where it has income and is paying a fair return. This is so astonishingly simple as a proposition that one constantly wonders why it should be disputed, yet, as a matter of fact, few appraisers have attempted to reason it out logically or compute properly the expense of financing a plant on a going basis, and the average estimator stops short when he gets to the end of his list of prices of visible things and wonders what he shall do next. It took a high Court ruling in the Kansas City Water-Works case, twenty years ago,* to compel the inclusion of the cost of reproducing the plant to an income-paying basis, and, ever since that decision, "going value" has been the bogy of valuators, utility commissions, and municipalities.

"Going value" is an unfortunate name. The Courts have attached those words to that part of the reproduction method which has to do with reproducing the financing of the physical property on a paying basis. As commonly used, it is nothing more or less than a necessary part of the reproduction process; as a principle, it is entirely simple and easy to understand.†

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* National Water-Works Company *vs.* Kansas City, 62 Fed., 853; 10 C. C. A., 653; 27 L. R. A., 827; 27 U. S. App., 165; July 2d, 1894.

† See "The Going Value of Water-Works," by Leonard Metcalf and John W. Alvord, Members, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 326 (1911); also "Notes on Going Value and Methods for its Computation," by John W. Alvord, *Proceedings*, Am. W. W. Assoc., 1909, p. 184.

As reproduction is a cost method, and as cost is not necessarily value, the term "going value", generally used to designate the completion of the cost-of-reproduction method, is a misnomer. It would be better to modify the term to "going cost", when speaking of that part of the reproduction which it describes.

When we have completed our reproduction-cost estimates, including going cost, and when we have reasoned from this line of evidence and other lines of evidence to the value of the property, then we might properly use the term "going value" to designate the entire property value as a going concern, for then we are talking about value and not about cost.

A great deal of perplexity in valuation work and valuation discussion arises from just such confusion of terms as is here described, and this perplexity will continue until engineers get more clearly defined notions of valuation work, for it appears to be necessary to use brief descriptive terms in such discussions.

Reproduction A Cost Method.

A great many estimators, and most of the utility commissions, use the reproduction method as though it were a sort of a formula for the final determination of value. In other words, they unconsciously assume that they are completing their valuation, step by step, with each item of the reproduction, and get afraid during the process that the final result will not be exactly what they want; that is, they fear it will lead them somewhere where their reason would not fully approve. All this would be obviated if the estimator had clearly in mind the simple proposition that reproduction is a cost method, and, as such a method, when completed it may or may not be the real value. When one has completed the reproduction estimates, he always has the right to reason whether it furnishes him with a good approximation of the value. This is why the Courts keep reiterating that other things should be taken into consideration, such as past cost, for instance, where it is available, market value where it can be found, and, more particularly, the surrounding conditions, such as the need for the service, the value of the service to the consumer, the adequacy of the service, and such other factors as have been heretofore mentioned. All these surrounding conditions, with the proper analysis, will show whether or not the reproduction cost method can be safely used, and

if not directly used, what modifying influences would properly be taken into consideration.

If, for instance, a plant has recently been built in a town the chief industries of which have unexpectedly come to an end, so that the town will inevitably depopulate, then the cost of reproducing such a plant is obviously not a proper measure of its value; or, if a new and better method of service be suddenly discovered or is soon likely to be discovered through new invention or change in the arts, then a similar result will ensue. On the other hand, if a plant is properly built, with reasonable skill, in a growing city, and has perhaps the most desirable kind of service, and is without engineering or financial errors, or dishonesty of management, or bad judgment in its financial development, then the value of that property may be even greater than would be indicated by the reproduction-cost method.

This matter of disassociating value from cost is one of the greatest difficulties that the engineering student of appraisal work encounters. It is so common an experience to most engineers to think of value as synonymous with cost, that the possibility of these two things being at times unequal is only received by the average mind with something akin to a wrench.

When an appraiser gets clearly in mind what reproduction is, that it is a cost method with many limitations, and that, although it is usually only one of the lines of evidence which should be taken into consideration, all difficulties with the logical details of reproduction will naturally and easily be resolved by the application of a little careful reasoning. It will be obvious that any doubts, for instance, about including pipe under pavements, interest during construction, ownership of services, overhead charges, preliminary expense, and going cost, arise mainly from the fact that the valuer has no clear idea of what he is doing or how it is to be useful when done. The valuer should never be afraid of a logical reproduction, but he should fearlessly and conscientiously follow it through to the logical end, not mixing it at any point with past cost, or deviating from the natural requirements of a practical and actual reconstruction in any way. When reproduction is thus carried out, the valuer should set it aside as one of the most important lines of evidence, and finally review it in connection with other lines of evidence and all the surrounding circumstances, many of which, though affecting value, cannot well be expressed in figures

at all, or can only be expressed partly in figures. His final conclusion as to what is the fair, just, and equitable value of the property in question, all things considered, should be reasoned out in the broadest kind of a way.

SUMMARY OF REPRODUCTION.

In reproducing a plant and property, therefore, we may summarize the following principles of procedure as in accordance both with the opinions of the Courts and as generally agreed to by those appraisal engineers of long experience in valuation work:

First.—The reproduction should be along lines that are humanly possible, and in accordance with practical experience.

Second.—It is entirely a hypothetical and conceptual process from beginning to end, and, therefore, to be practically useful, must be founded on a long and wide practical experience.

Third.—It is to be based on a practical working plan, in which the time adopted for the proper procedure is one of the most important elements.

Fourth.—A hypothetical reproduction, to be practical, must be predicated on a reasonable construction and development period in the near future.

Fifth.—The costs for this near future must be based on the prices of the recent past, just as any engineer entering into any new work determines the probable near future costs from past experience.

Sixth.—The reproduction is not complete until the plant is financed on a paying basis, *i. e.,* earning a fair return.

Seventh.—To use this reproduction as a measure of the present plant, the depreciation of the existing property must be computed and allowed for.

Eighth.—Reproduction is a cost method, and, as such, may or may not be of value. When reproduction is completed up to a paying basis for the property, then, and not until then, have we a right to reason whether this cost method is a good measure of value.

Ninth.—If a plant is carefully and honestly built, if it serves a need in the community in the best possible manner, and if there is no better or less expensive service, then the cost of reproduction must of necessity be the least possible value that can be put on the utility, but the real value may often be in that case more than the reproduction cost.

Tenth.—On the other hand, if a plant has been recklessly overbuilt, or if the need for its service has diminished, or changes in the arts require its early abandonment, then the cost of reproduction will largely exceed the true value, unless materially reduced by an adequate allowance for depreciation of a general nature.

Past Cost.

It is interesting to note the tendency of the newer public utility commissions to emphasize past cost data as of major importance, and substitute it in whole or in part for reproduction cost wherever possible. Almost all appraisers have been through this experience. Later and more mature consideration generally converts them to giving more weight to the reproduction process. This tendency to emphasize past cost is not in accord with the opinions and reasoning of the higher Courts, which are constantly pointing out that it is the value of the property now that must be sought, and, though the Courts do not deny that past cost is a proper and suitable line of inquiry, they very properly insist that it is only useful because it may throw some light on the value of the property now. This line of thought of the Courts may be summarized, as follows:

First.—Past cost often does not include all the actual cash cost incurred in promoting the enterprise, nor does it usually show the deficits which have occurred below a proper revenue from time to time.

Second.—It lacks any indication as to how the property has appreciated, by reason of its own growth and, as well, the growth of the community in which it is located, or the growth of demand for its product from a given community.

Third.—It usually contains the cost of all obsolete structures, and, as found on the books, has usually no allowance for depreciation.

Fourth.—There is nothing to show the hazards in the original installation. Plants were often originally built when enterprises of a similar nature were new and untried, when the community was small, and the value of public service was unknown and unappreciated.

Fifth.—Markets for special classes of supplies, machinery, and labor, have fluctuated widely, and in some cases radically, so that any attempt to reason present cost from past cost must inevitably be a task of great speculative difficulty.

Sixth.—Granted that we could properly compute all the depreciations and the appreciations, and could determine correctly the proper deficits in revenue and make proper allowances for changes in the markets for material and labor, and compute correctly the hazards and cost of money under the original circumstances, we would not even then have a measure of value, but only a measure of cost, which might correspond with reproduction cost now, but this might or might not be the value of the property, as the case might be.

It is seldom possible to find an accurate and complete record of past cost, intelligently kept on a proper accounting basis, with proper separation of operating from capital account.

When the past cost of a public utility company is available and can be found from the books, it usually appeals strongly to the buyer side of the question, for the simple reason that it is often less in amount than the present value of the property, and usually less than the actual cost and the present cost by reproduction methods. Cases arise, however, where past cost is much greater than present value or replacement cost.

Undoubtedly, Courts, economists, and engineers would all like to have as definite and satisfactory a method on which to reason value as past cost would seem to be on the face of it, but, unfortunately, its face appearances are most misleading, for if we must find the value of the property now, that value is probably less controlled by past cost than any of the other cost methods from which the value can be reasoned.

The procedure adopted by some utility commissions, of utilizing reproduction methods for the structure cost, and adding to this amount past deficits in operation to produce a total, has always seemed to the writer to be singularly devoid of any rational basis of explanation, for it mutilates the reproduction of a property so as to render it illogical and incorrect. Some comments on this procedure may be summarized, as follows:

First.—It appears to be the result of a timidity about the reproduction method, which it is believed would not exist if it were fully understood that reproduction cost is not an arbitrary measure of value.

Second.—It leaves entirely uncompleted a valuable line of inquiry, that of reproduction logically carried out to its logical completion.

Third.—It does not encourage the summing up of the real present reproduction cost and the actual past cost as two entirely distinct and valuable lines of evidence, each to be used as a basis for further investigation and reasoning.

Fourth.—It appears to have resulted from an over-anxiety to reason about value before all the cost evidence is in hand.

Fifth.—It tends to discourage the adequate consideration of important factors which influence value, and which cannot always be well expressed in figures, and this effect is caused by reasoning on value at too early a stage in the proceedings.

Sixth.—It would seem that the consideration of past deficits is given too much prominence, because their inclusion in present value puts a premium on poor judgment or misfortunes in the past. The best managed and most valuable properties, under this method, would show the least value, and the most unfortunate and worthless would show up the best.

Seventh.—It would appear that past deficits belong to the past cost line of evidence, and are properly considered there, and not in reproduction cost evidence.

Eighth.—It is quite impossible to see how deficits of past operating cost have anything to do with the cost of financing a physical property into a paying income now.

MARKET VALUE.

Although the Courts have said that market value is one of the elements to be considered in valuing a utility property, as a matter of practical application, it is found that a broad and representative market for the securities of a utility company is not usually found, except in the case of the railways and the larger utilities in our great cities. In most of the smaller utilities a general market either does not exist for the property as a whole, or is so restricted as to be an unsafe guide as to its value.

In the process of reproduction, however, we apply the principles of market value indirectly. We may not find that the value of the property, as a whole, may be determined in this way, but by the process of an imaginary re-creation we may find the market value of materials and labor, machines and appliances, and so utilize market

value indirectly for at least a portion of the more tangible parts of the property.

In the larger utilities and the railways, where stocks and bonds are regularly listed and broadly dealt in, market value has more meaning, and should be reviewed, bearing in mind, however, its fluctuating character and its attempted prophetic indication of future rather than present values.

The market value of to-day is almost always built up on the hopes or fears for the morrow, and thus, to a large extent, may consist of too much optimism or pessimism for real present usefulness.

Market values, as indirectly introduced in the reproduction method, are also productive of embarrassment, for it is certain that the value of utility property, as a whole, does not fluctuate with the variations of the material and labor market on the detailed items of which it is composed, and this should always be remembered when the final reasoning is in progress.

On Reasoning From Cost to Value.

By far the most important part of the matter of determining a just and reasonable value of a property lies in summing up all the evidence obtained from cost or other methods, and reasoning from these facts and estimates to the broader question of value of the property, as a whole.

Singularly enough, this important phase of valuation work has not been generally appreciated by engineers, but it has been constantly emphasized or discussed by the Courts. Most engineers assume that the process of valuing is finished when the cost evidence is complete, and by that time they usually have become so beclouded by the mass of detail through which they have struggled that they are often mentally unfitted to undertake the broader and more important responsibility of reasoning to the final result.

It is true that if engineers are seriously to value a property they must familiarize themselves thoroughly with the details of that property, by carefully working out such lines of cost evidence as are necessary, but it by no means follows that they must neglect the broader aspect of the question when this preliminary and detailed work is done.

The general summation of value, from all the surrounding facts, is the part of the problem where the mental methods of the broad-minded business man and financier come into play, and it is precisely here that the business man becomes impatient with the Court and the engineers. Large buyers of public utility properties purchase such properties without any preliminary replacement estimates whatever. What a business man generally wants to know is the character of the city, the needs of the population, its enterprise and probable future growth, the presence or absence of business and manufacturing industries, and, as well, any probable competition or probable changes in demand. He wants to know the gross and net revenues actually existing. He will look carefully at the possibilities of economies, and especially at the probability of increasing the demand for the product or service. From these lines of information, and possibly comparative per capita costs of similar enterprises elsewhere, he will rapidly frame up in his mind a maximum and minimum value within which he can afford to trade for the plant, and the point at which the willing seller meets the willing buyer finally fixes the value of the plant in terms of money. Now, what appraisal work lacks to date is more of this kind of final review, well reasoned out after all the cost lines of evidence are in and complete.

To a man of affairs, accustomed to this kind of rapid valuation, the methods of the Courts, the appraisers, and the utility commissions look quite absurd—how absurd may be well indicated here by a rude illustration:

Appraising A Horse.

When an ordinary farmer or stockman wants to purchase a horse, he runs over in his mind the prices at which, ordinarily, horses have been selling, the uses and needs he has for this particular horse, the earnings the horse can make for him in the uses he will put it to, also the kind of a horse he feels will suit the conditions, and then he fixes, in his mind, the maximum amount he can afford to pay for such a horse, based on its usefulness, and looks for a willing seller having about the kind of a horse he wants.

Now, let us suppose the earnings of horses were regulated by law to only a fair return on their value. It would become necessary, under such circumstances, to seek a process which would show us what a

horse would be worth that could fill a particular need, at a regulated rate of earnings; the rate, in turn, being predicated on the value of the horse. This would reduce our stock farmer to a condition of mental coma, as it would introduce economic problems that are outside the domain of his practical experience, and he would probably have to seek the advice of a specialist.

Let us suppose that one of our modern, up-to-date utility commissions, with its engineering staff, were called in to help out, and suppose that it should proceed as we proceed to-day in the valuation of private property devoted to public use. First of all, it would probably take off the horse's shoes and weigh them, and compute the market cost of replacing them; then it would calculate the quantity of glue in the hoofs; get the measurement of the hide, and compute the fertilizer, estimate the grease, and value all the by-products at prices at which they could be purchased in the open market, and after all of this was done, it would total up the cost of reproducing the "physical property," so-called, and then perhaps suddenly discover that it had now only arrived at the cost of replacing a dead horse.

The problem would then be to find the added cost of producing a live horse, over and above the cost of a dead horse, and here the commission is in difficulty. The suggestion would occur to it, of course, that the hypothetical dead horse which it had just estimated, might be compared with the live horse which it is valuing, and the difference in cost might be called "going value," but this would not look very sensible. First of all, it would have to assume the cost of a live horse, which is assuming one of the results that are being sought.

Another method, following along the lines of the reproduction method, would be to give the carcass oxygen treatment, and, if possible, animate it into a live horse, which, after forced feeding and stimulants for a few days, might be able to do some work and earn certain revenue. The cost of all this biological proceeding might be compared, in the meantime, with the earning power of the live horse so as to arrive at "going cost," and be added to the carcass estimate. Some of the commissions would prefer to go back and see what it would cost to raise a horse from a colt, keeping track of the cost of feed, pasture, and care, crediting the horse with the work performed, and thus arrive at the net cost of the full-grown horse. Others would add the early deficits in earnings of the colt to the carcass value of the present dead horse, for

which they have completed estimates, and arrive at the net cost of a live horse in that way. Most engineers, new to appraisal work, would jump at the difference in cost between the dead and the live animal, but try to conceal the fact, that they have guessed it, under a mass of verbiage, which, when analyzed, is found to be evasive and meaningless. Once having arrived, by hook or crook, at the cost of a breathing working horse, most of our appraisal authorities stop, quite exhausted with such strenuous endeavor in these new fields of inquiry.

Now the difficulty of finding the going cost of a live horse, over and above the carcass cost, may seem far-fetched and not properly comparable with other kinds of property, but is it so far-fetched after all? What is it that goes into an incongruous mass of stone, brick, iron, and machinery that makes it a harmonious, useful, operating whole? Is it not brains, thought, patience, perseverance, foresight, ability, human energy, confidence, and wisdom? And these are but attributes of life; the characteristics of live things over and above dead things. When we make a property live, therefore, for useful purpose, we put something of the Divine human into it, which is little short of miraculous, and the operation of measuring this in terms of cost is necessarily a difficult and delicate task to which we must give careful thought, not only to the physical task of bringing a property to life, but to the psychological and ethical task of making it live usefully.

The stockman, however, might properly want to know what kind of a horse it was, after the appraiser had got it alive. Should it be classified as a race-horse, a carriage-horse, a cart-horse, or only just a jackass—and it might make a good deal of difference in earning power, as well as value, whether it were the first or the last?

Courts and utility commissions do not always disclose their mental processes, and, so far, we find on the records but little reasoning along the broader business lines which affect and surround the question as to whether the given utility is a race-horse or a jackass. Engineers, especially, are quite content if they can only get at the cost of a breathing, living horse, but it is pretty evident that this is an important question, and should be given a good deal more consideration in the future than it has had in the past.

Under the restricted methods of procedure, in which purely cost methods have such predominance, some utility properties have undoubtedly been undervalued, others must have been overvalued, and

unless we can come to appreciate the fact that surrounding conditions, good judgment, and skillful engineering tell in the value of a property, we shall not get away from this danger.

Now, the Courts and commissions being charged with the responsibility of determining values, which, in part at least, are dependent on rates, and at the same time of establishing rates which are a fair return on the value of the property, are under the most insidious temptation to lessen property values without appearing to do so, and without even intending to do so. This is due, in large part, to the unfamiliarity of the Courts and commissions with the business considerations which control the investment of capital and its expectation of profit, and also to the pressure of the public clamor as the buyer of service. Where appraisal authorities have yielded to these temptations, and particularly where they have failed to give due consideration to all the surrounding conditions outside of the cost evidence which go to make up value, they have, in some cases at least, placed the racehorse on a level with the jackass, and perhaps at times elevated the jackass to a dignity to which it was not entitled.

As has been said before, where a property is serving a real need in a community that is growing normally, and where its installation and management are without serious errors of judgment, the value of the property cannot be less than the cost of reproducing it in a manner that is humanly possible, less its depreciation. Ordinarily, it is worth more, and usually it would be considered poor financial judgment to sell a well-designed and well-established property in a growing town for the bare cost to reproduce it, were it not for the power of the Courts and the commissions to restrict value through the establishment of rates.

This brings us to the consideration of the economic law that when value depends on rates, rates must be such as to attract capital into the utility field or there will be no value commensurate with the necessary service; yet it is to be doubted if any actual rates so far established are of themselves high enough to account for the fact that capital is apparently continuing to move into certain kinds of utilities, especially into some of the municipal utilities. What are in reality the main inviting causes here may be summarized, as follows:

First.—The belief that the fundamental law of this country will protect property devoted to public use from confiscation.

Second.—The belief that legal supervision by non-partisan commissions will decrease the annoyance such investments have been put to in the past, and make the capital involved more secure.

Third.—The demonstrated fact that earnings from public utility investments do not fluctuate so greatly in financial depressions as do some other forms of investment.

Fourth.—The fact that the demand for the service will usually grow or can be encouraged to grow at a faster rate than the increase in population.

Fifth.—The probability, or at least the hope, that the value of the property in a public utility will grow with the community, irrespective of the actual investment in money.

Sixth.—The obvious intention of the Courts and commissions to treat such property fairly in the matter of reasonable rates.

So long as capital finds these conditions satisfactory, it will flow into the utility field in a selective way, but should these fundamental conditions be seriously altered or much changed for the worse, capital will grow timid, and will gradually withdraw.

In the case of privately owned municipal water supplies, for instance, the consumer public has, in the past, greatly overestimated the profits and underestimated the necessary value of the property required in the service, and, as a result, has so harassed such utilities that capital for many years has not only declined to enter the field, but has sought every opportunity to withdraw from it, even at a sacrifice. A further important instance illustrating this timidity of capital is noted at present in the steam railway field, where the adequacy of the present rates has come of late to be seriously questioned, and, as a result, capital has been less willing to enter the field during the last few years, with consequent higher cost to the railroads for money.

In reasoning from cost to value in public utilities, it is desirable to recognize that capital has properly treated the public, not only by good service, but by that further attention which wins and retains their good will, not that this of itself is an element of value, but it undoubtedly confirms and strengthens other elements of value.

On the other hand, we must take into consideration the fundamental treatment of capital by the public. If this is to be hostile and without intelligence, then values of all kinds will be lessened, and the

service will be ultimately destroyed. If it is to be scientific and just, then values will be maintained, and the service can be made efficient.

Now, the public is vitally and directly interested in the maintenance of values, just as capital is interested in having the public's good will, for without values we cannot have the use of capital, and without the use of capital we cannot have the reasonable service at fair rates, and we need and want the service.

In reasoning from cost and other evidence to value, however, there must be found some way to reward sound judgment, skillful engineering, and economical management, producing good service. This, none of the present utility laws as interpreted by the commissions, does; in fact, the present utility scheme of regulation, as now administered, distinctly puts a premium on poor or indifferent management, and takes away the incentive for economy. The next great problem in utility regulation will be to find ways of overcoming this defect.

In the meantime, there is no simpler way of recognizing good investments and careful management, where they exist, than by recognizing their effect in increased plant values, and this should be a part of the final reasoning in every valuation, just as much as indifferent judgment, absence of good engineering, and lack of economy should be similarly dealt with in the final summation.

It will always be very helpful if the maximum and minimum limitations to value be one of the matters to be reviewed before finally determining value, and, for this purpose, it is desirable, in the interests of the public, to reason out what would be the results of natural and economic conditions that would exist under a free competition that it is not wise actually to permit, and is not, in fact, intended to be permitted under our utility laws as they are now framed.

The average buyer in the open market always has his maximum price in mind, just as the seller has his minimum price, all of which is helpful in arriving at an exchange value. Helpful reasoning can be introduced along these lines in the final summation of public utility values. For instance, it would not be wise, on the one hand, to reward good judgment and skillful engineering in a property by fixing a value well above that which in an open and competitive condition would excite (if it were possible) the increased competition of a similar kind of supply or service, or the construction of a rival or duplicate property; nor, on the other hand, could we penalize dishonest financiering

and thoughtless construction by reducing values below the point where any service would be ultimately impossible, and the public finally be made to suffer.

Reasoning from the cost and other evidence to value, therefore, is a financial as well as a judicial and ethical problem, and if we would here do justice to the subject we must imitate the methods and habits of thought of the honest but alert business mind. When this economic study is followed through, it is striking to note how identical at every point are the common interests of capital and the public interest.

CONCLUSION.

There is probably no problem presented to the American public at the present time which is so difficult and complex as that of valuing the public utilities of this country. The value of these utilities is constantly fluctuating. It is dependent on many complex surroundings, and involves many difficult and delicate questions. To determine lines of evidence which will show the value of these properties in a way which the higher Courts have indicated, often involves the engineer, the economist, and the Court, in a complex hypothetical proceeding, which is only rescued from being absurd by being applied by competent and fair-minded men, with long practical experience behind them. The foundation for all this work undoubtedly rests in the law and its interpretation by the Courts, and the superstructure only is the field of the engineer and economist. Let us, therefore, as prudent men and experienced engineers be careful that we do not foolishly build on false premises and produce results which are "confusion worse confounded."

Once having founded ourselves properly as to principles, but one other great problem remains, that of holding to the judicial attitude. An entire paper might profitably be written on this subject alone. The engineer in appraisal work, to be successful, must keep himself unbiased and unprejudiced. At times he must resist the popular clamor for low, unjust, and unfair valuations, and at times he must resist the temptation to become the willing tool of powerful interests.

The preliminary bias in approaching public utility valuation has a great deal to do with progress in solving its difficult questions. Roughly, there are six classes of minds that ordinarily approach the problem of public utility valuation:

First.—Minds which are only familiar, by sentimental interest, with the rights of the public, and care little or nothing about learning about the conservation of property.

Second.—Minds which are fully familiar with the rights of the public, but are also open-minded and desire to learn about the conservation of property.

Third.—Minds which are, by training and environment, only familiar with the rights of property, and are not open-minded about learning anything about the rights of the public.

Fourth.—Minds which, by training and environment, are only familiar with the rights of property, but would also like to be unbiased and know more about the rights of the public.

Fifth.—Unbiased minds which are interested in just relations between the public and private property interests, but have not fully analyzed both viewpoints or had experience with both situations.

Sixth.—Minds which are naturally unbiased and are interested in and have mastered the just relations between the public and private property interests.

Classes 1 and 3 are worse than useless as appraisers, and, as writers, serve only to obscure and befog the problem of the just relations between the public and private property. They, too, often produce heat when there should be light. They are usually hopeless, and should not be entrusted with judicial functions in utility valuations.

Classes 2 and 4 may or may not develop into Class 5 or 6, depending on their inherent fairness and opportunity to study the opposing side of the question. Usually, they do not get enough opportunity to study the reverse side of the question until their opinions are too firmly fixed to be altered.

Class 5 may develop into Class 6 if opportunities come for sufficient experience and study on both sides of the question. They have analytical minds and judicial natures in the making.

Class 6 can only come from an unbiased and analytical mind that has had ample experience in representing both sides of the question.

To produce good results it is necessary, in appraisal work, to have practical experience in considerable amount, not always representing one side, but alternately representing both conflicting interests, in

order to appreciate fully the varied points of view which can be brought to this difficult matter.

Cross-examinations in court, conferences with able attorneys, familiarity with the views of Courts and commissions, arguments with opposing valuators, experience on various kinds of appraisals: these all develop the practical, fair, and unbiased appraiser.

Nothing is so illuminating as to find oneself demonstrating value from two opposing viewpoints in two consecutive appraisals. A few experiences of that kind give one confidence in one's own sense of justice.

There is one golden rule for the engineering appraiser in valuation work:

Always value a property so that you would be satisfied with your valuation were you appointed by the other side at interest, and so that you will be satisfied with it ten years hence, as well as satisfied with it next month, if you actually find yourself on the other side of the issue in some other case.

APPENDIX

Legal Decisions.

If the opinions advanced in this paper are well founded, then engineers who propose to fit themselves for valuation work should by all means make a careful study of the opinions of the Courts. What is greatly needed is a book, compiling in full, without partiality, the decisions and opinions of the Courts in valuation cases, all of which should be accompanied by a topical index. Probably this will be produced and published at some early date, as the importance of the subject now urgently requires that it be done.

In the meantime, a list of the more important decisions, especially those of the Supreme Court, is appended to this paper for immediate reference. The list may be profitably extended, but it is believed that most of the leading cases are included:

Ames *vs.* Union Pacific Ry., 64 Fed., 177; Digest, 2, 13, 26, 34, 38; Abstract, 48 to 50.

City of Bristol *vs.* The Bristol Water-Works Co., 23 R. I., 278, 49; Atl., 974.

Brymer *vs.* Butler, Pa., Water Co., 179 Pa., 250.

The Brunswick and Topsham Water District (Me.) *vs.* The Maine Water Co., 99 Maine, 371.

Baker *vs.* Gertside, 86 Pa., 498.

Chicago Railroads *vs.* Day C. C., 35 Fed., 866.

Railroads *vs.* State of Minnesota, 134 U. S., 418-456.

The Cedar Rapids Water Co. *vs.* City of Cedar Rapids, 91 Northwestern, 1051.

The Cedar Rapids Gas Light Co. *vs.* City of Cedar Rapids, 120 N. W., 966.

Cotting *vs.* Kansas City Stock Yards Co., 183 U. S., 79.

Contra Costa Water Co. *vs.* City of Oakland, Cal., 113 Pac., 668.

The Consolidated Gas Co. *vs.* City of New York, 157 Fed., 849-855.

Covington Gas Co. *vs.* City of Covington, Ky., 22 Ky., L. R., 796.

The Des Moines Water Co. *vs.* City of Des Moines, 192 Fed., 193.

The Galena Water Co. *vs.* City of Galena, 87 Pac., 735.

The Gloucester Water Co. *vs.* City of Gloucester, Mass., 60 M. E., 977.

The City of Janesville, Wis., *vs.* Janesville Water Co., R. R. Comm., Vol. 6, 628.

The Kennebec Water District *vs.* Waterville Water Co., 97 Maine, 185.

The City of Knoxville, Tenn., *vs.* The Knoxville Water Co., 212 U. S., 1.

Little Falls Electric and Water Co., 102 Fed., 663.

Monongahela Navigation Co. *vs.* U. S., 148 U. S., 312.

Munn *vs.* Ill., 94 U. S., 113, 133.
National Water-Works Co. *vs.* Kansas City, 62 Fed., 853.
Newburyport Water Co. *vs.* City of Newburyport, Mass., 168 Mass., 541; 47 N. E., 533.
The Norwich Gas and Electric Co. *vs.* City of Norwich.
The City of Omaha *vs.* The Omaha Water Co., 30 Sup. Ct. Rep., 615.
Peck *vs.* Chicago R. R., 94 U. S., 164; 24 L. Ed., 97.
The Pioneer Tel. and Tel. Co. *vs.* Westenhaver.
Queensborough Gas and Elec. Co. *vs.* Pub. Ser. Com., N. Y.
Redlands Water Co. *vs.* City of Redlands, Cal., 121 Cal., 365.
Stanislaus County *vs.* City of San Joaquin and Kings River Co., 192 U. S., 201.
San Diego Water Co. *vs.* City of San Diego, 118 Cal., 556.
San Diego Land and Town Co. *vs.* Jasper, 110 Fed., 702.
San Diego Land and Town Co. *vs.* Jasper, 189 U. S., 439.
San Diego Land and Town Co. *vs.* National City, 174 U. S., 757.
Slocum *vs.* City of North Platte, Nebr., 192 Fed., 252.
Smyth *vs.* Ames, 169 U. S., 466.
Spring Valley Water Co. *vs.* The City of San Francisco, Cal., 124 Fed., 574.
Spring Valley Water Co. *vs.* The City of San Francisco, Cal., 192 Fed., 137.
Spring Valley Water Co. *vs.* Schlotte, 110 U. S., 347; Sup. Ct., 4851.
Trust Co. of America *vs.* City of Rhinelander, Wis., 182 Fed., 64.
C. H. Vanner *vs.* Urbana W. W. Co., 174 Fed., 348.
Washington Gas Light Co. *vs.* District of Columbia, 161 U. S., 316.
Willcox *vs.* Consolidated Gas, 212 U. S., 19-54.
Wilkesbarre *vs.* Spring Brook Water Co., 4 Lack., Legal News, 567, 380.
Western Union Tel. Co. *vs.* Myatt, C. C., 98 Fed., 335-342, 354.
Bothwell *vs.* Consumers Co., 13 Idaho, 568, 9e Pac., 533; also 24 L. R. A. (U. S.), 485.
Minnesota Rate Cases, 230 U. S., 352.

DISCUSSION

Mr. Haupt.

LEWIS M. HAUPT,* M. AM. SOC. C. E. (by letter).—This paper is an able and timely exposition of a vital question affecting the public weal and concerning which the Engineering Profession has much to do.

The author has stated the fundamental elements clearly, as being legal, economic, and constructive, and has given the definitions necessary to reach clear and just conceptions of their relations to one another as well as the constitutional basis of an equitable settlement for private property taken for public use, under the right of eminent domain.

Yet it seems there are other factors not so readily ascertained, which enter into the solution of the just value. These factors are psychological, and are influenced largely by the relations of those who take part in such adjudication. These the author has classified under six heads, the first three of which he rejects as useless and the next two as of doubtful expediency, depending on the trend of their bias. The sixth is the only condition under which one would be apt to render an impartial judgment, that is, the "unbiased and analytical mind that has had ample experience in representing both sides of the question."

In other words, the judge must be competent to determine present values of property as well as franchises, and be a follower of the Golden Rule. Thus, there is introduced the fourth fundamental element of morality as over against policy, which is so often a dominant factor.

In practice how often does it happen that the evidence is tainted with the desire to win the case at all hazards, or that the witnesses—employees—are suborned by the fear of losing their positions if the truth be told, on the theory that "whose bread I eat, his song I sing"; or the impression that to depreciate value and force the holders of private property to sell at a bargain-counter price is a popular measure and will be sanctioned by the public at large, or that the Government consists of those executives who may be temporarily in power and who are paid to drive hard-and-fast bargains with the people, even to the destruction and confiscation of personal or corporate property, at public expense, which the Fifth Amendment was expressly designed to prevent.

One has only to examine the records of the acquisition of the private waterways and other utilities of the country, or on the Isthmus, to note that the first effect of such a measure is to depress the market values of the securities of prosperous "going concerns", and that in some instances official appraisements are made on depreciated unit prices or reduced quantities of materials, and claims are made for services, or threats that free, competitive, parallel routes will be con-

* Cynwyd, Pa.

structured out of the public treasury, at much greater cost, unless the parties who have invested their funds in good faith and are dependent on returns of interest for their subsistence, consent to surrender for less than half the appraised value, as has recently been attempted. Mr. Haupt.

Surely such methods cannot receive the sanction of honorable engineers much less of a righteous nation, yet they have been proposed but have failed in some instances.

In a democracy, such as this, it would seem that what is just for the whole people as sovereign should be equally just for each individual unit, and that if a statute runs against the individual or the corporation, it should also hold against all the people, who are "the Government." The monarchical fiction holds that "the King can do no wrong" and, therefore, excepts the sovereign from the penalties which it compels the individual (who is far less able) to pay, and he has no redress at Court, and relief by legislation is a vain hope.

A Definite Method.—As to the difficulties of ascertaining the present worth of a public utility, it may be apropos to call attention to the acquisition of the property of the Monongahela Navigation Company which, for many years, was a private enterprise furnishing a means for providing the great manufacturing districts in and around Pittsburgh with coal, at very low figures, until it was thought best to emancipate the tolls by having the Government acquire the property and operate it at the expense of all the people. The argument presented was that the Government had developed an open waterway on the Kanawha in competition with the Pittsburgh District which was subject to tolls and paid a tax to the State for its franchise.

The Company was inaugurated under a Pennsylvania State charter on June 14th, 1836, and developed "an immense commerce." The Act provided that the works might be purchased after 25 years by paying the expense of construction and repairs, with the net dividends, with 8% interest thereon. The work was actually completed in 1884. The Government Board found (in 1895) that "the money value of the commerce is very great and far in excess of that of a majority of streams upon which the United States, under its policy of fostering commerce, has expended millions of dollars in improvements."

The President of the Company states that "previous to the recent agitation of the question of purchase by the United States the stock sold at a premium of 80 per cent. * * *, making the present value of the Company's improvement, on a basis of 4½% interest, over \$4 000 000." Concerning which the Board states that the number of shares and their market value being variable, it was unable to fix on any amount which properly represents such value at any future time.

Mr. Haupt.

The books of the Company show the cost of construction and repairs to have been $3 033 531.34. The Board found "the present intrinsic value of existing works about $1 950 000", exclusive of the franchise.

On October 12th, 1886, President Moorhead stated to the Board, "we wish to be left in the undisturbed possession of our property, and protest that no public interest can be subserved by its appropriation

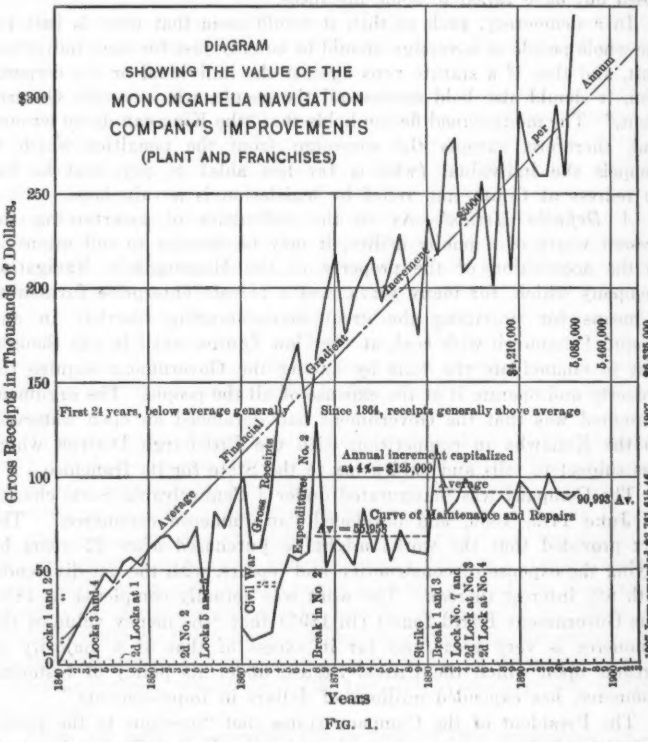

FIG. 1.

by the United States; that the cost of coal to the consumer will hardly be appreciably diminished, while the annual charges to the United States would be burdensome and increasing, as permanent repairs would be required * * *. It would constitute class legislation of the most objectionable kind. The rates of tolls are the lowest charged on any canal or slack-water improvement in the United States and so low that to take them off altogether would not lessen

the price of coal 1 cent per ton * * *. The Government cannot take this property without making compensation, which is the full value of the property of the Company." Mr. Haupt.

No dividends were declared between 1839 and 1853, during which time the construction was in progress.*

These negotiations were continued from 1884 to 1907, when the final award was made, by a jury, for the sum of $3 761 615.46, after 23 years of delay. At this date the value of the works and franchises had increased, as shown by Fig. 1, to about $6 335 000, if capitalized at 4% on the average annual increment of $5 000 net. By this method the actual financial status of the utility is clearly set forth, as it shows the gross receipts from the capital invested, the cost of construction, maintenance, and operation; the influence of social, physical, and commercial agencies during the life of the enterprise and the average net returns, so that, under the doctrine of probabilities, the curve of future value at any given date may be closely approximated.

It eliminates, also, to a very great extent, the personal equation or erroneous judgment due to the desires of the parties at interest, either of buyer or seller, as it deals only with the facts as measured in dollars and cents and, therefore, should commend itself to all parties who may be retained to ascertain the "just compensation" which should always be paid when a private utility is to be acquired for public convenience.

In another instance, the Government has been negotiating for an important strategic canal route, of great utility, for more than 40 years, concerning which the Chief of Engineers has reported that it would save $1 229 242 on the actual commerce now using it, and "probably considerably in excess of this amount", so that, to effect such an economy (regardless of risks and insurance), it would be justifiable to expend more than $30 000 000, if capitalized at 4%, and yet it hesitates to invest even one-third of this sum for the development of our domestic commerce. Surely an enterprise of such great utility should commend itself for immediate development.

L. L. Jewel,† M. Am. Soc. C. E. (by letter).—This paper has been read with a great deal of interest and admiration. It is a masterly presentation of a difficult and complicated subject, and could have been made only after a close and unbiased study of the relations between private interests on the one hand and community interests on the other. Mr. Jewel.

In studying the lines of evidence of value which are summarized on page 141, one is impressed with the unimportance, or rather, say, the indirect character, of some of them. For example, the market value of the securities of a property will very rarely, and may never,

* Ex. Doc. No. 249, H. R. 53d Cong., 3d Sess., Jan., 1895.

† Saranac Lake, N. Y.

Mr. Jewel. have any direct relation to its value. Even when there is a broad, free market for them, and the state of mind and of finances of the investing public is normal and undisturbed by any of the thousands of factors that may and do disturb it, the market value of the securities of a property will, to quote the author's apt phrase, reflect more "the hopes or fears for the morrow" than the value of to-day.

Again, the values of other properties similarly situated will seldom be available, and, when they are, could hardly be used except for purposes of comparison with a value already fixed by direct evidence.

Finally, it is hard to see how past cost necessarily has any relation whatever to present value. It is a measure of the efficiency—perhaps the honesty—of promotion, of construction, and of operation; but it certainly can be no dependable measure of present value, nor necessarily have any determinable relation to it.

It would seem, therefore, that the lines of evidence of value as given may be separated into two groups or classes: one containing the lines of direct evidence of value, and the other the lines of evidence which are always of secondary and often of no importance in determining value, but which, nevertheless, may be to some extent useful as a check or balance on the result obtained from direct sources. That is to say, the just value of a property for any purpose in which the consuming public is interested is directly dependent on two factors:

(1) The cost of reproduction, starting with a naked idea and finishing with a going concern, as it is;

(2) What may be called a business estimate of value, depending on the many factors and conditions peculiar to that particular property and community as given by the author in the paragraph called "Fourth" on page 141.

A value thus determined could hardly be influenced, although it might be checked, by the other lines of evidence given.

Such a classification of the author's lines of evidence may be criticized as encroaching on the judicial function of weighing the evidence, and as being outside the scope of the paper. It seems to the writer, however, to be not only in harmony with the ideas expressed throughout the paper and with the foundation laid down by the law, but also to be of universal application, and therefore a fundamental principle of public utility valuation.

Mr. Dana. RICHARD T. DANA,* M. AM. SOC. C. E. (by letter).—The author, as he says himself, has assumed an ambitious task, and has produced a work that, if only for the emphasis so skilfully laid on the necessity for dependence on legal principles, cannot fail to further materially the development of scientific methods of appraisal work. The bibliography of legal decisions is timely and valuable. Perhaps People *ex rel.*

* New York City.

Kings County Lighting Company *vs.* Wilcox *et al.*, and the Spokane Rate case, might profitably be added. The final paragraph of the paper, defining what the author calls a Golden Rule for the engineering appraiser in valuation work, is more than admirable. Mr. Dana.

Some of the definitions on pages 120, 121, are hardly as comprehensive as would seem to be necessary, in view of the fact that they are intended to establish a definitive basis for the development of principles heretofore somewhat obscure. Two, in particular, will be mentioned: "Property" and "Value". "That which belongs exclusively to an individual" is, in the writer's judgment, a rather inadequate description of "Property". Property connotes proprietorship, and proprietorship may be exercised by a corporation, a firm, a Government, or a State, etc., each of which is distinctive, and may not be an individual. "The right of possession" seems to be hardly in the same generic class as the thing possessed, and is objectionable to the writer on that account. Webster's Dictionary recognizes both meanings of the term, but takes pains to separate them.

Economists have striven to define the term "value" satisfactorily, with but poor success to date. "A measure of the relation of two services growing out of the adjustment of mutual needs" implies, to the writer's mind, a good deal of abstruse theory. In its fundamental concept, and postulating labor as the basic principle of wealth, this definition may be theoretically correct, but, for practical purposes in appraisal work, it seems to be too complicated. The other definition of this term, given by the author, seems to be hardly adequate, as the undesirable qualities certainly affect the value, and should be considered.

Any definition of value which presupposes a free market, a willing purchaser, a perfect and inflexible medium of exchange, is necessarily too restrictive. The science of economics needs for this term a better definition than any which has appeared thus far. Value is not cost, it is not price. For appraisal purposes, the value of a property might perhaps be called the quantity of its net assets, physical and non-physical, all expressed on the same basis of currency.

On page 124 Mr. Alvord deprecates a "Jack of all Trades", particularly in appraisals, but he does not make it exactly clear, at least to the writer, just who ought to do most of the appraising, the "experts from several professions", or "engineering specialists from each type of utilities to be valued".

In every public utility corporation of considerable size there are usually several very able engineers who are thoroughly familiar with the property because of long connection with it and intimate association with its problems. These men, under the general direction of an engineer who has specialized in economics, costs, and appraisals,

Mr. Dana. will make the best possible "team" for the solution of the composite problems that are inseparable from any appraisal, as has been proved many a time and oft. It has also been proved, to the tune of many millions of dollars, that the appraisal specialist need not be a specialist in electricity in order to make the best appraisal of an electric plant. Some of the worst appraisers of railroad property have been railroad men, some of the best painters have been poor judges of the value of pictures painted by others. To appraise an egg properly, one need not have been a hen.

Moreover, a specialist in economics, and of general engineering training, can find values in one kind of property to which he is led by knowledge of analogous values in another kind of property, which would be overlooked without such general experience.

Granted equal age and intelligence, the man who has had experience of appraising utilities of many kinds will make a better appraisal of any one kind than will he who has confined himself to that one kind of utility all his life, without opportunity to adapt the ideas gleaned from a wider field. This principle, indeed, is fundamental in all managerial processes, of which appraising is not the least.

Mr. Lavis. F. Lavis,* M. Am. Soc. C. E.—The Society and the Engineering Profession are to be congratulated on this valuable addition to the now somewhat voluminous literature on the subject of valuation. This discussion of the fundamental principles, and the clear definition of the points which can be defined, as well as the discussion of those about which there is still some doubt, will do a great deal toward placing the subject on a somewhat more rational basis than that on which it has hitherto seemed to rest.

The whole paper will well repay careful study, but there are some points which the speaker believes may be especially emphasized:

First.—That, although the reproduction method seems to be most generally applicable, there can be no absolutely hard-and-fast rules laid down to govern the methods to be used. It seems most important to point out that each case must be considered on its merits.

Second.—The necessity of collecting and properly arranging all the facts and data relating to each individual case before attempting to formulate opinions.

Third.—The six items cited as necessary to remember, in weighing evidence as to value, especially the first, referring to the fact that value can never be absolute, but is always a matter of intelligent opinion.

Fourth.—The very vivid description (on page 144) of the development process. Most particularly apropos being the reference to

* New York City.

the "easy and virtuous hindsight" that tells us how the property might be reproduced by supermen, instead of causing us to remember some of our own very human lack of foresight which, to some extent at least, has increased, in some way, the cost of every piece of construction with which we have been connected. Mr. Lavis.

Fifth.—The imperative necessity that engineers must be guided by the law as it exists and as it has been interpreted by the Courts. This, however, though eminently desirable, need not cause us to forget that both the law and its interpretation are subject to changes, and are sometimes at least viewed "in the light of reason" which we might possibly be able to make shine.

In the enumeration (on page 141) of those items which must be considered in order to arrive at the cost of reproduction, Mr. Alvord is very careful to point out specifically, in Item *D*, the necessity of estimating the depreciation, but he does not call attention to the equally important necessity of including also the appreciation. It is true, of course, that an intelligent estimate of the cost of reproduction in itself would cover the estimation of both appreciation and depreciation, and, therefore, both may be considered to be covered by Item *C*; but as long as Item *D* was thought to be a necessary supplement to Item *C*, it would appear to be desirable also to include a further note in regard to appreciation, especially as this is so much more likely not to be fully allowed.

There is probably a question as to how far engineers may go toward the determination of the complete value of any live operated property, especially of transportation systems. Mr. Alvord very appropriately points out the necessary co-operation of those experienced in the law, and in economics or finance, as well as engineering, in arriving at this final result. As far as the railways are concerned, it is equally necessary to obtain the aid of those skilled in operation, in traffic and its sources and destinations, and in other allied features, as they usually have a far more adequate idea than engineers as to the costs of development—a most important item of value.

The speaker is almost entirely in accord with the views expressed in the paper, and believes that, speaking generally, they apply equally well to all valuation. The paper, however, is distinctly written from the viewpoint of their particular application to the circumscribed and unified types of utilities, such as lighting and water supply, or even electric railway plants, rather than to steam railways, and this must be kept in mind. This is particularly noticeable in the statement that it is a "demonstrated fact that earnings from public utility investments do not fluctuate so greatly in financial depressions as do some other forms of investment." This most assuredly does not apply to steam railroads, but is generally true of utilities supplying individual municipal needs.

Mr. Lavis.

As far as these two general types of properties are concerned, there might be little difference in the principles governing their valuation were it not for the question of the use which it is proposed to make of such valuation when it is made. Valuations are made for many purposes, but the one which is of the most vital present interest is the use of this value for the purpose of the regulation of rates. It is when considering the values of properties for this purpose that the railway situation exhibits difficulties which do not seem to present themselves when considering isolated and complete units.

It is hardly necessary to quote individual cases or examples, as these will readily occur to those having experience in the railroad situation; it does seem worth while, however, to call attention to this point, and to the natural corollary, that therefore neither the market value of securities nor the past or present earnings can be used in the determination of values for this purpose.

Mr. Tribus.

LOUIS L. TRIBUS,* M. AM. SOC. C. E.—Many of the current articles on utility valuation give evidence of idiosyncrasies, but Mr. Alvord's paper should be considered as a very able, interesting, and comprehensive presentation of rules and methods of arriving at values, together with much basic information and many common-sense comments.

In the speaker's earlier days, the then older engineers, trained largely in the "School of Hard Knocks", were looked up to admiringly, for from them emanated an atmosphere of good sense, and in their infrequent writings and discussions were found the epitomes of judgment, founded on sound principles, because they were the result of well-digested practical successes and failures.

Oft-times, these same men could not, or would not, formulate the theories on which their works were built, or deduce conclusions from them.

Engineers were but little bound by precedent, so that individual thought had fairly free rein.

This is not telling of the remote past, but facts prevailing within very recent years.

About 15 years ago, virtually began the great consolidation of allied interests and an awakening to the possibility of increased efficiency in management; valuation work as now known was then born, though only a crystallized phase of a principle prevailing from the days of Cain and Abel. Unfortunately, with the extreme swing of the pendulum, "efficiency" has become a word more nearly for ridicule than admiration, because many, even excellent, engineers have been led astray by the allurements of blank forms and colored inks, substituting a phantom for reality, and valuation as a newer hobby may easily reach the same goal.

* New York City.

Mr. Tribus.

Socialism is now the ruling doctrine of America, even if we do not fully acknowledge it, and unless level-headed men will interject common sense into public thought, there is no knowing to what extremes it may lead. The present public attitude toward public utilities is a marked example.

Engineers have been receiving an increasingly thorough technical training, though perhaps too much specialized at the start, and writers like Mr. Alvord have, sometimes with great clearness, made available for even the tyro, thorough discussions of subjects not found in text-books and almost unknown to the teacher of a few years past. The teacher's good sense, however, often put into the mind of the student the broad principles which enabled him to make good selections of mental tools and meet the successive problems of practice with credit.

The socialistic idea, however, has already secured dominance in our larger cities, in the legislatures, and even in the Courts, with some phases to the advantage of the public, but not all.

This is nowhere more prominent than in the treatment of the public service corporations. Where is the group of capitalists willing to-day to invest in a new to-be-privately-owned water-works system? There is no safety in such ownership, with the restricted franchise, the low-rate, short-term hydrant contracts, subjected to the whims of constantly changing city officials, virtual confiscation of property at the end of the franchise period, and constant supervision and regulation of rates by public service commissions. Engineers are not wholly free from blame in producing that effect.

In the speaker's hearing, a well-known engineer objected to a certain city paying to a small water company any material sum for its plant, because "the city could not use it as a part of the larger system, which it would construct at some indefinite time, when it got ready." The company had not had a successful financial career, but had made possible the growth and development of the community served, bringing to the city, in increased taxes, far more than the interest on any sum that might have been required to purchase the plant. In many a valuation case the parties representing those in power have indicated a similar sentiment, though rarely so shortly or brutally expressed.

The horse trade referred to in the paper is the typical basis of all valuations for sale or acquisition of a public utility, except that, when a community is the purchaser, it wields an absolutely unfair advantage, for it can enter into ruinous competition and collect in the tax levy all losses, while the private owner goes steadily into bankruptcy, or accepts the offer made, whatever loss may be entailed.

Granted that arbitration of some form is resorted to, the owners are placed in the position of a defendant who has to prove every item of value rather than have his claims disproved by the community, consequently often many items of real value and legitimate expense are lost in the summations.

Mr. Tribus. How are proper figures to be reached? Here comes the value of such theories as described by this paper and others somewhat similar.

Applying several of the methods will give several differing resultants; now for the horse trader's reasoning: The horse may live for so many years, he may encounter no accidents or illness, therefore he will be able to make good return on the purchase price; the would-be purchaser fixes $200 as the reasonable value, but might pay more, yet cannily offers $175. The seller thinks along the same lines, but remembers some veterinarian's bills, a large feed account, and some lost time, yet also has in mind some excellent earnings; he thinks of $275 with longing, but $225 rather than run risks, and asks $250. Finally, $225 is reached by dickering. No rule of Court based on comprehensive valuing would have brought that result, and thus is it with more important work. The expert evidence worked out on various bases produces an effect on the mind of the jury or Court, but judgment—or sometimes averages—determines the final decision.

If the arbitrators wish to convey the impression of great learning and very careful weighing of the evidence, a "dictum" accompanies the "opinion", and a "rule of Court" is born, to be quoted for all time as a guide in other cases, to be followed, perchance, as sheep follow a leader.

Socialistic sentiment is at heart founded on absolute justice between man and man, but socialistic action and legislation is more the result of the one-time-oppressed squaring accounts (and a little more) with the oppressor.

The public utility operator of the past, as a class, has not served the public with strict justice and consideration, and so is not entirely free from responsibility for the judgment often meted out to-day.

All the more reason why the engineer called into a valuation case shall be fair in his advice, as well as intelligent and comprehensive in his study.

That brings us to the point that is too often overlooked: no two cases, generally speaking, are alike or will bear the imposition of the same rules.

It is incumbent, therefore, for the engineer to make a study of the local conditions that affect the utility, its origin and financing, its construction, its operations, its effect on the community, its financial rewards or, perchance, losses, its obsolescence, whether due to poor original judgment, or to radical improvements in machinery, methods, etc., in fact, all the many factors which enter into success or failure.

Then, applying such methods as seem to have applicability, figures will be derived that can have some force in determining the final "horse-trade" judgment.

Without following certain illustrious examples, in condemning the Courts, quietly, engineers may remember that at one time many decisions happened to favor those in litigation who were represented

by particular lawyers retained by the party having the longest purse, but to-day votes are more apt to prevail, the decision going to interests wielding the balance of power. Mr. Tribus.

Sometimes non-residence ownership enters the problem, not that the engineer should consider it a factor, but to look out for it and forestall injustice by the thoroughness of his presentation of facts and deductions.

In this discussion the engineer is considered as a judge, not as an advocate; of course he may advise counsel as to points favorable to his side, but on the stand he must be absolutely impartial; and a righteous judge will give great weight to the testimony of a just engineer. We would not intimate, either, that the unrighteous judge is in the majority; something in the nature of such a trust brings out the fair qualities of a man, if they are in him, but too often, influence is stronger than right.

Valuations for rate-making, and taxation, are simply different phases of the same problem, only giving to a possibly injured owner a little greater chance for later redress through review.

The one point that the speaker wishes to emphasize strongly is the futility of rules for reaching decision as to value, though their benefit is unquestioned for suggesting a range of values, which may be somewhere near the truth; more important than all, however, is a thorough study of all the different factors in light of prior experience as to the financing, designing, constructing, and operating problems involved in the utility under discussion.

The engineer without practical experience is as valueless in handling such questions as the lawyer would be in designing the utility itself.

Let us have less of precedent, fewer differentiations of rules and dicta, and more horse-sense in valuation work.

Joseph Mayer,* M. Am. Soc. C. E. (by letter).—This admirable paper clears up many of the confusions pervading the discussion of the valuation of public utilities, but leaves some points still in a fog, which arises mainly from the author's assumption that law and the decisions of Courts, and not justice and the nature of things, are the supreme authority in valuation. Law is but an imperfectly expressed authoritative opinion of the legislature attempting to define justice, and must be and is construed so as to produce justice or such relations among men as will secure the public welfare. The public welfare, and not the Constitution, is the supreme law of every land, as the Romans, the creators of the first great system of laws, already knew. Mr. Mayer.

The author is aware that the idea and nature of value is the foundation of all valuation, but refrains from giving an explicit definition of the idea of value that forms the foundation of his paper. Instead,

* Montreal, Que., Canada.

Mr. Mayer. he gives us a miscellaneous assortment of legal and economic contradictory definitions of value, none of which is quite adequate as a basis for the discussion.

He gives us:

"*Value.*—The sum of the desirable qualities which render a thing useful and sought, measured in money; a measure of the relation of two services growing out of the adjustment of mutual needs."

These are two contradictory definitions. The sum of the desirable qualities which render a thing useful and sought cannot be measured in money because the amount of money which such a sum commands depends on other independent factors. Air is essential to life, and therefore presents a very large sum of desirable qualities, but its value measured in money does not measure this sum. The second part of this definition, which contradicts the first, is more nearly true, because value is a measure of relations between many services, no two of which define value. It is quite indefinite, however, because there are many different relations between services. The value of a thing or service is a measure of the quantities of many other things it will buy. Money is one of these other things. The quantity of money which can be obtained for a thing is its price. Money at a given time and place has a definite value which can be more easily ascertained than the value of most other things, and its value is more constant in time and space than that of most other things. Therefore, it is the most convenient approximate measure of value. The value of money, however, sometimes fluctuates rapidly; the price of things becomes then a very unreliable measure of their value.

The author also gives us:

"*Market Value.*—The adjustment of two services, *i. e.,* as between a willing seller and a willing buyer under open conditions of competition."

This defines the ratio of the values of the two services which would prevail if competition were universal, but it does not define the competitive value of either; many comparisons are needed to define any kind of value of anything. Since monopolies are spreading, the prices and values which co-exist with monopolies are of most practical importance. If the clause "under open conditions of competition" means simply with competition between many independents, not co-operating buyers and sellers of the two services, then the definition gives the ratio of their values under the given conditions. This ratio it is possible to ascertain if the described conditions actually exist.

Another:

"The sum of money a thing will produce to the seller when sold."

Mr. Mayer.

This is the price, not the value.

Another:

"The quantity of labor and capital necessary to produce a given article, or, in other words, the actual cost of its production, is the true criterion of its worth."

This is a clumsy attempt to define the cost of production, not the value. The price of the labor and the interest on the capital used in the production of a thing, together with the profit of the owners and the rent of the land used, with the taxes and other incidental expenses, determine the cost of production, but not the value of a thing which can be produced. Many valuable things, like land and genuine antiquities, cannot now be produced, and therefore have no present cost of production, but they have value.

The author does not state which of these contradictory definitions he believes to be true, and, throughout his paper, does not adhere strictly to any one of them. He has in his mind a much better definition of value, resulting from his wide experience in valuation, which is more nearly correct than any one of those he gives, to which he generally adheres, and which creates the great value of his paper, but he sometimes abandons it in deference to the Courts, and then he becomes confused and obscure.

The Courts are not authorities on logic. The principles of logic are above the Courts, and demand that the realities must govern all definitions. There are groups of things and groups of phenomena in the external nature; the things or phenomena in each group have certain features in common. These groups are separate from each other; there are few or no transitional forms or phenomena between them. The definitions must draw the lines where Nature has drawn them. True definitions can only be given by those who know thoroughly the things or phenomena they attempt to define. Economists, and not Courts, are most competent to give the true definitions of value which must serve as the basis for valuations. Courts, however, are competent to construe the laws and to decide which values in the possession of individuals can, and which cannot, be taken by the public without compensation. The attempts of the Courts to define values have mostly the purpose to distinguish between these two varieties. It is quite proper to define different varieties of values, but the definitions should describe groups of realities which actually exist in the external world, not only in the mind of the reasoner, and which have real limits that separate them from other groups; and nothing should be called a variety of value which does not belong to the species value.

Court definitions which describe actually existing varieties of value are practically important for valuators, and must be considered

Mr. Mayer. in their proper place. The Courts mention fair and reasonable value and fair and reasonable return on the same, without giving definitions of these terms. Their meaning must be inferred by studying the decisions based on them. Until they have been so defined as properly to describe actually existing groups of value which have definite limits separating them from other groups, they cannot be used as the basis of a logical discussion. Beside the nature of value, the laws which govern the changes in values are of the utmost importance in valuation.

Man desires to obtain certain things; he will produce them, if this is the easiest way to get them, and if he likes them more than he dislikes the trouble of producing them. For some things he will make great, for others only small, sacrifices or efforts. The amount of effort or sacrifice he is willing to make, if necessary, to obtain a thing, measures for him its apparent usefulness, and this is the upper limit of the value of the thing for him. The actual value of the thing, as long as he lives alone, is the effort he must make to get it; this is also its cost of production. Things of the same value, under these conditions, have the same cost of production, and things of the same cost of production have the same value, provided their apparent usefulness is at least equal to their cost of production. The usefulness does not measure value, it only fixes an upper limit which value cannot exceed.

The usefulness may also be defined as the degree in which a thing serves the needs of life. It is, however, not the actual but the apparent usefulness which is measured by the amount of effort one is willing to make to obtain a thing. Things like land, which cannot be produced, can be acquired by effort, and the necessary effort to acquire them measures their value if this does not exceed their usefulness. These are the laws of value, only as long as a man is alone. Other laws prevail where many different men live together and exchange things with each other. If all men were exactly alike and had the same possessions and surroundings there would be no reason for exchanges, and the laws of value above described would also hold for a society of men.

In a society of different men with different possessions and surroundings, the cost of production of the same thing is different for different men. Each will now tend to produce those things or services for which he is best fitted, and will exchange his surplus of these things or services with others to obtain those which he can only produce with great effort or not at all, to the profit of all.

Markets then arise where the prices of things are fixed so that the efficient demand for them equals the supply. The efficient demand for a thing at a given price is the quantity asked for by those willing to

pay the price. The supply is the quantity offered. Both change with the price. Mr. Mayer.

The supply of such things as cannot be produced is necessarily not governed by their cost of production, but, if they are owned by many who do not co-operate, it is governed by the prices and by their usefulness to their present owners. If all the owners co-operate, such a quantity only may be put on the market as will bring the largest total price. A small quantity may then bring a larger price than a large one. Where all or most of the producers of a thing co-operate to limit the supply, higher prices are secured than with free competition among many independent producers.

The supply of such things as are produced by many independent, not co-operating, producers, tends to increase when the average profits of their producers are larger than in other productive industries, because capital will flow into the most profitable channels.

Competition is disappearing in many lines of production, and especially so in public utilities. With the disappearance of competition, other laws begin to govern prices and profits.

Without regulation, the prices of the products of monopolized industries are much higher than would be required to secure the same average profits as in competitive industries. The difference between monopolized and competitive profits goes largely to promoters and reorganizers of monopolies. Capital invested in unregulated monopolies is generally highly profitable. The promoters and reorganizers obtain the required capital from the public by selling stocks and bonds at prices which bring to the buyers often only about competitive profits. If the consumers of monopolized products organize and fix such prices for them as will give but competitive profits, they can get the required capital from the public, and the prices will be the same as if competition existed. The purpose of the regulation of monopolies is to obtain monopolized products at competitive prices. There are two great difficulties to be overcome in such regulation. The first is to avoid injustice to the present owners of the stocks and bonds of monopolies; the second is that of securing efficiency of management. Valuation is mainly intended to overcome the first; the second, thus far, has been but little considered.

The Courts endeavor to do justice to the owners and the public by giving to the owners a fair return on the present fair value of their property.

The author endeavors to ascertain this fair value of the property of monopolies, and consults for this purpose the Fifth and Fourteenth Amendments of the Constitution of the United States and the decisions of the higher Courts.

The two Amendments prescribe that no State shall deprive any person of life, liberty, or property, without due process of law, and

Mr. Mayer. that private property shall not be taken for public use without just compensation. To grasp the meaning of these clauses we must know what is due process of law, in taking private property, and what is just compensation. This we can find by observing closely how private property is taken for the State or Nation. The legislatures are constantly engaged in changing values and thereby taking and giving property to private persons. Legislation might be defined as a process of changing values so as to secure the public welfare thereby. When a tariff law is passed, the values of many goods extensively imported are changed, and with them the values of the factories making them. If States introduce prohibition, the values of breweries and distilleries in these, and to some extent in other, States, are reduced. If a State fixes minimum rates of wages, it changes the value of labor. All tax laws change values and take property. No compensation is ever given by the Courts. It is evident that the Courts either consider these values not to be property, or they believe that no compensation is here just compensation, or that a manufacturer knows, when he enters a protected industry, that the protection may be removed by the legislature and that the value of his plant and the income from it is influenced by this knowledge to such an extent that he receives thereby compensation for the risk. This, however, introduces an element of gambling into such industries, which is objectionable. The Courts would probably not sustain this if it were practicable to remove it. These instances are given to show that it does not follow from the Constitution that compensation will always be given where private property, or at least a part of its value, is taken by the State or Nation. They show that the public welfare and the nature of things are the supreme law. The security of property from aggressions by the State, therefore, is not absolute unless all these values are not considered property. It is not important for our purpose whether the values which we consider are or are not property. What is important to know is, which values can and which cannot be taken by the State without compensation. The Constitution gives us no answer to the question. Therefore we must go to the decisions of the Courts in similar cases, and to the principles implied if not expressed in these decisions.

The author has endeavored to find that value which the Courts will award to the public utility companies, and he gives the method he follows in ascertaining it. The variety of value awarded to these companies must be a group having a real existence, and it must be within the species value. All kinds of value are defined as the quantities of other services or things a service or thing will buy; this defines the species value.

Under different conditions, any kind of service will have different values. Varieties of values can only be defined by defining the condi-

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tions which influence their value. One variety is that value which exists for the lonesome producer. Another variety exists where there is universal competition, another where unregulated monopolies exist, and many others with regulations of various kinds. The most important condition influencing the value of regulated monopolies is the kind of regulation adopted. The Courts insist that the present value is the fair value to be ascertained. The leading lines of evidence to ascertain the present value are, according to the author, a statement of the earnings and expenses during recent years; the market values of the securities of a property where a broad and representative market exists; the cost of reproduction less depreciation, past cost, and other factors. The Courts and the author neglect to mention the most important factor influencing the present value of a public utility, namely, the kind of regulation to which it is to be submitted in the near future. They are attempting to determine the present value of these utilities as if it were independent of the regulation to be adopted, they practically assert that the value can be determined without knowing what regulation will be adopted. This is a mere legal fiction, in the minds of the author and the judges, which contradicts the facts. There exists no variety or kind of value which can be ascertained without knowing all the influential conditions which determine it. To illustrate: The value of an old house is found mainly by estimating the future net revenue which will be obtained by the owner, and estimating the present value of this estimated net revenue; if, however, the value at a near-by future date of the then future net revenue differs largely from the present value of the future net revenue, the former has an influence on the present intrinsic value. The intrinsic value of a house is that value which would prevail if the buyers and sellers would all correctly estimate this future net revenue and the present market value of it, and if they were all governed in their buying and selling by these correct estimates, and by similar estimates for near-by future dates. This intrinsic value the valuator tries to find. It is a definite thing at any time, it does not change rapidly, though the value shown by actual sales may fluctuate widely above and below the intrinsic value. The actual buyers and sellers are governed by more or less erroneous estimates of the future net revenue, based principally on the present net revenue and a consideration of the causes which will change it in the future; they may also consider causes which change the prevailing rate of interest. If the interest rate is rising, the present value of future payments decreases; this reduces the value of the house in the future, other things remaining the same. The knowledge that the value is increasing or decreasing also influences the present value. The buyers and sellers will also consider the ratio between values and net revenues prevailing in the same neighborhood for similar

Mr. Mayer. property; they find an approximately constant ratio between the two and then multiply the estimated net revenue by this ratio to obtain the value. If the house considered is out of style and out of harmony with the demands made by the possible buyers of such houses, he will not try to find the cost of production of a similar new house and the depreciation by obsolescence. He will estimate, from the present net revenue and the causes of change, the future net revenue and its present value; or he may ascertain the cost of production (including the cost of land) of a new house adapted to present tastes which gives the same present value of the future net revenue.

To make this clearer, consider the value of a large electric power-plant of, say, 100 000 h.p., with reciprocating steam engines. Such plants are now built much more economically with steam turbines. The value of the old plant is the present value of its future net revenue. The future net revenue depends largely on the future price of power. The price of power, under competitive conditions and without regulation, is mainly governed by its cost of production by the most economical methods known. The cost of a steam turbine power-plant which can obtain the same future net revenue as the old plant with reciprocating engines is approximately the value of this old plant, because plants giving the same revenue have the same value, and the value of a modern plant is, under competitive conditions, approximately equal to its cost of production. It would be most absurd to attempt to find the depreciated value of the old plant by finding first the cost of reproduction new of a similar plant and then the difference between this cost and the value of the old plant. The similar new plant would be antiquated, and its cost of production does not indicate its value. Its value could only be found by ascertaining the cost of production of a modern plant bringing a future net revenue of the same value. After this the values of the antiquated new and the antiquated old plant would have to be compared by comparing the present values of their future net revenues. Since the cost of reproduction new has no influence on the value of an antiquated plant, it is a waste of time and money to determine it. The cost of reproduction new of an old antiquated plant, therefore, is absolutely worthless as an evidence of value. In all cases where obsolescence is an important factor, the only rational method of finding the value is by comparison with the cost of production of an equally valuable modern plant, or one producing a future net revenue of equal present value. Any cost of production, however, is only under competitive conditions an approximate measure of value. Where monopolies prevail, the cost of production does not measure the value. With regulated monopolies, the value depends mainly on the kind of regulation adopted. As the author states, the value of a well-designed plant

meeting the needs of a growing market is at least equal to the cost of production under any kind of practicable regulation; but the value may be much more with the existing methods of regulation. The recent net earnings are only a leading evidence of value when they indicate the probable future net earnings; this they do only if the regulation in the future is the same in its effect on the value as that in the recent past. Mr. Mayer.

The author in his method of valuation practically assumes that this regulation will be the same in its effects on value as the regulation of the recent past; therefore he believes that the fair value of a public utility is that value which exists if the future regulation produces the same values of the present properties as that in force in the recent past; and he consistently follows this definition, except where he is compelled by deference to the Courts, for the purpose of obtaining their approval for his valuations, to deviate from it. As the author states, the Courts are very cautious, and cannot be induced to commit themselves to wide generalization; this is the function of the legislatures, not of the Courts, and the latter should give a clear definition of the fair value of utility companies; then they could be valued by ordinary business methods of valuation without insisting on the exact ascertainment of the value of the carcass for the purpose of obtaining the value of a live horse.

The author says that valuation consists in collecting the facts influencing value and in judging them. He is practically compelled to omit the most important fact—the future regulation—and he can only describe superficially the various operations to be performed in the judging, but he cannot find a real method of judging without first, implicitly, if not expressly, assuming what kind of value he desires to ascertain. A clear idea of what you want to ascertain is absolutely necessary for finding the methods of ascertaining it. The present value of public utilities is very nearly identical with the value which would prevail if the future regulation were the same as the present one. Substantial justice, or as close an approach to justice as practicable, would be done to the present owners of the securities of these corporations if their value were ascertained with this assumption, and if a fair return on this value would be paid to the stockholders. This, however, must not be decided by the sense of justice of any one, or any class of individuals, but by the will of the whole community expressed by the legislatures or the Courts. Most of the conflict of opinion in the discussion of valuation arises from conflicting ideas of justice. With universal competition, fair prices and values rise, if each producer is left free to offer his products at whatever price he pleases. Under such conditions the average profits in all industries tend toward equality, therefore the idea arose that justice requires that each should be free to choose the prices at which he is willing

Mr. Mayer. to sell or buy. After natural and artificial monopolies became numerous and important, this absolute freedom of action did not any more produce just prices; the old ideas of justice, however, were for a long time enforced by the Courts and legislatures, and great injustice in the distribution of income and wealth resulted therefrom. This brought about a conviction that such absolute freedom cannot be allowed, and efforts were made by the legislatures, the Courts and the, for this purpose created, public utility commissions, to establish just prices. Under the old laws very large returns on the invested capital were secured in the monopolized industries, and their value, therefore, often largely exceeded their cost of production. Whether the values of monopolies created under the mixture of freedom and regulation of the recent past shall belong to the owners of the corporations or partly to the public is still unsettled. Similar changes of conditions and of prevailing ideas of justice characterize all historical development, as is shown by Henry Sumner Maine.* He says, in his chapter entitled "Legal Fictions":

"In progressive societies social necessities and social opinion are always more or less in advance of law. We may come indefinitely near to the closing of the gap between them, but it has a constant tendency to reopen. Law is stable and some societies progress. The greater or less happiness of a people depends on the degree of promptitude with which the gulf is narrowed."

Much of the difficulty in valuation arises from the insistence of the Courts on their legal fiction that it is possible to find the value of a regulated public utility company without knowing the nature of the regulation. This necessitates the abandonment of the only correct and reasonably accurate business men's method of finding the value of a public utility company by estimating the present value of its future net earnings; and forces the adoption of some artificial and extremely complicated substitute that pretends to find its value without considering the regulation, and that is nevertheless compelled to assume implicitly a definite kind of regulation, as it is quite impossible to solve the problem in any other way. No agreement on questions of valuation is possible until either the Courts change their course or the legislatures settle the question by statute law.

Another important legal term requiring a clear definition is "a fair return". For the stockholder of public utility companies, this is just as important as the meaning of "fair value", unless he is offered cash payment for the fair value of his property. It makes an enormous difference whether 5 or 8% is considered a fair return, and this should not be left to individual judgment, or merely guessed at, but should be determined on such known general principles that the investors can foresee the decision. There is substantial agreement,

* "Ancient Laws."

however, that a fair return for any monopolized industry is the same return on its value as is obtained in competitive industries, or a return just large enough to obtain the necessary capital; but there are individualists and socialists defending two radically different methods of distributing the profits of a monopolized industry among the different enterprises engaged in it. The individualists say the regulation should be such that the profits in every enterprise will be the same as they would be if competition existed. With free competition the profit in every individual enterprise is different; some bring large dividends, some go into receivers' hands, some are entirely abandoned on account of changes in the arts, but the average rate of profit in every industry at the same place and time tends to be the same. There are differences of average profit at different times and places. In a very new country, profits are larger than in the East. The average rates of profit at different times and places vary mainly with the rate of interest, and they are often larger in growing than in decaying industries. To bring about such a condition of things in the monopolized industries, it would be necessary to ascertain the average rate of profit in competitive industries and to establish such prices of monopolized products as will produce in each monopolized industry the same average rate of profit as in the competitive industries at the same time and place, with slight variations to bring about the needed increase or decrease of supply. The prices of gas and electricity, the street car fares, etc., would have to be different in different places. The differences would have to be equal to the differences in cost of production with ordinary methods of production, as is approximately the case in competitive industries. The profits of the different enterprises would then depend mainly on the efficiency of management, or the amount of intelligence, conscience, and energy put into each enterprise, and the same inducements to secure efficiency would exist as with free competition. Mr. Mayer.

The writer has roughly sketched such a regulation in a previous paper.*

The public utility companies may endeavor in their regulations to approach this ideal of different profits in proportion to efficiency of management, or they may set before themselves another ideal, namely, equal profits in all enterprises, large enough to secure the necessary capital for increases of plant. If this latter ideal were followed consistently the stockholder would have no pecuniary inducement to select efficient directors, and the directors chosen would have none to obtain efficient management; or to adopt the most judicious salaries and wages for all the employees and to employ the best men obtainable for them; or to watch the performance of every one and to advance

* "The Just Value of Monopolies, and the Regulation of the Prices of Their Products", *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 455.

Mr. Mayer. them for merit only; or to study all proposed improvements and to adopt them quickly if meritorious. All these things would have to be obtained through the supervision of the public utility commissions. The directors would become useless puppets, and they would soon be discarded as an unnecessary incumbrance. If the ideal were followed consistently, the stockholders would become practically bondholders, and money could be obtained at a moderate rate of interest, either by the sale of nominal stock or of bonds. The risk would be carried by the public, and the management would become entirely responsible to the public instead of to the stockholders. The public would become the virtual proprietor at the moment the ideal was adopted and consistently followed. Good results can be obtained by public ownership with public management, but not by public ownership and risk with private management.

This ideal of regulation is not yet followed consistently by the Courts and the public utility commissions, but they are gradually being compelled to adopt it by the manner of fixing the prices of monopolized products.

The consumers or one or more of the utility companies complain that the prices are too high or too low. A Court or a public utility commission is requested to fix fair prices. They proceed to value the property by some extremely artificial and therefore inaccurate method, which must hide the fact that some kind of regulation is necessarily assumed in the valuation. After the valuation is obtained, a rate of profit is fixed which is believed to be just adequate to secure the necessary capital for extensions, and the prices are adjusted to produce this rate of profit. The risks of investments in controlled public utilities are peculiar and entirely different from those in competitive enterprises. Therefore it is very uncertain what rate of profit will make it possible to obtain the necessary capital for extensions. The Courts generally let a master decide this knotty question; the public utility companies guess at it themselves, as this is the best they can do under the circumstances. To fix a just rate of return for average efficiency of management, it would be necessary to ascertain the average rate of return in competitive enterprises and to create such conditions for the monopolized industry that the risks of capital are the same as with free competition. To do this to settle an individual dispute is impossible. An intelligent guess is therefore the best practicable solution of the problem. The commission or Court would undoubtedly fix a lower rate of profit where they find legal evidence of gross inefficiency or dishonesty, or might reduce grossly excessive salaries, but, for most of the factors which really govern the efficiency of management, no legal evidence can be presented to the Courts or commissions, and therefore they cannot be considered; but, even where such factors are legally known, it is impossible by the available

machinery to calculate or estimate correctly the amount of change in cost of production caused by them and to allow for it by a different rate of profit. Mr. Mayer.

The purely mental qualities of intelligence, honesty, skill, and energy, which cause such large differences in the cost of production of the same goods, selling at the same prices in competitive industries, and consequent large differences in profit, cannot be directly measured; they can only be judged by the material effects they produce. The Courts and commissions, by the now prevalent methods of fixing prices of monopolized products, are practically compelled to ignore all differences in such mental qualities which cannot be measured by their method of procedure, and to award equal profits to all enterprises, or equal compensation to intelligence, honesty, skill, and energy, and to stupidity, dishonesty, incompetence, and laziness. The Courts are established to settle individual disputes, and they cannot deviate from their usual method of considering each dispute separately from all the others. The utility commissions must either abandon this method, by creating such just systems of prices as would prevail if free and fair competition did exist; or they must obtain the capital for monopolized industries at a fixed and consequently low rate of interest; and they must manage these enterprises themselves, discarding the private boards of directors elected by stockholders. Until one of these alternatives is chosen, increasing inefficiency and expensive capital with consequent high prices are inevitable. Statute law must decide which of these alternatives should be chosen for the different monopolized industries.

Philip Burgess,* M. Am. Soc. C. E. (by letter).—The writer has read this paper with great interest. It is a very timely and valuable contribution to the literature on the subject, and will be appreciated by any one interested in the matter, and especially by any one who has had occasion to follow through a particular case, as developed by hearings before a Court or commission. That members of the Engineering Profession frequently do not have a clear conception of the fundamental principles involved in the subject is very apparent from their testimony before Courts and from their discussions of papers, as printed in the publications of this Society. Such differences of opinion can be accounted for only by the fact that engineers have not taken the time, or have not had the ability, or opportunity, to acquaint themselves with the present status of the subject of the valuation of utility properties, as determined by decisions of the Courts. It is apparent, moreover, in many instances, as indicated by Mr. Alvord, that engineers frequently appear to know more about the subject than do the Courts, a position which, of course, leaves Mr. Burgess.

* Columbus, Ohio.

Mr. Burgess. the witnesses open to severe criticism, or even to ridicule, and has a tendency to lessen the value of all expert testimony, even when given by qualified witnesses. This has undoubtedly called forth the frequently cited remark by a Court to the effect that expert testimony is the most unsatisfactory of all evidence. Fortunately, this criticism is not confined to engineers.

The writer has had the pleasure of listening to Mr. Alvord's testimony when on the witness stand for several days, and has been much interested and impressed by the fact that he always has had a clear conception of the matter at hand, as he saw it, and has always started with some fundamental conception of the principle involved, and then has endeavored to follow this conception through to a logical conclusion. It is undoubtedly true that there is no subject before the engineer to-day which requires so clear a conception of the fundamental principle involved as does the appraisal of properties devoted to public use.

One instance may be cited to show how much the value of the testimony of an engineer may be weakened unless he has a clear conception of these principles. In a water-rate case, in which the writer was interested, a certain witness for the utility—who is a member of this Society—submitted a report on the cost of reproduction new of the properties in question, and, when the report was handed in as evidence, stated that he did not know whether or not the cost of cutting through pavements over pipe lines should be included, and that such cost was not included in the report. Subsequently, Mr. Alvord, also called in by the company as a witness, stated why, in his opinion, the cost of cutting through pavements should be included. Later, the first witness asked to be allowed to state that he now believed that the cost of cutting through pavements should be included.

Under such circumstances, one can but ask the question as to whether or not the witness would have made up his mind on such an important item so quickly had he been on the other side of the table. It is very apparent that the witness did not have a clear conception of the fundamental principles involved, or that he was prejudiced.

Such instances of disputed items, whereon, perhaps, the Courts have not given deciding opinions, arise in all cases of valuation. It seems to the writer that, unless the engineer wants to appear in an unfavorable light, he must have a logical reason for arriving at his conclusions in all matters pertaining to the case, or he must state that "he does not know". He may well qualify such a statement to the effect that the matter is a legal and not an engineering question, and in fact is one for the Court itself to determine. If the Court decides that the item in question is a proper one to include, the amount

may be stated to be so much. The point as to whether or not the item is a fair one to include may far better be argued by the attorneys in the case than by the engineering witness. Mr. Burgess.

Doubtless one of the most valuable features of Mr. Alvord's paper will be found to be the legal citations and references with which, of course, any engineer who pretends to qualify for appraisement work should be familiar.

Mr. Alvord has indicated, in characteristically clear language, the four fundamental lines of evidence to be considered in making an appraisement. These are: (1) the cost of reproduction new; (2) the market value of securities of the company; (3) the comparative value of similar properties; and (4) the past costs of the properties in question. The writer believes that he is warranted in assuming that the importance of each of these items, in the author's opinion, is indicated by the order in which they are placed. In fact, in two instances in water-rate cases wherein the author and the writer have been jointly interested, although on opposite sides, the author has been contented to adopt his conclusion, as reached by investigations of the cost of reproduction new, without any apparent investigation into the actual past cost of the properties or of many items comprising them.

Mr. Alvord's paper may be considered as a very able argument, or demonstration, of the value of the cost of reproduction new as the foremost line of evidence to be considered in arriving at the value of public utility properties, especially if we include his qualifications of such costs new as indicated in his fourth line of evidence to be considered, namely, the relations of the utility to the consumer, the value of the service to the consumer, and the general business status of the properties.

It would also seem to be apparent that Mr. Alvord is very much disappointed that some appraisers have seen fit to give so much value to such evidence as past cost of the properties. As the author very well states: "What does it profit us to discuss the details when this serious and fundamental difference exists?" The writer believes that, after all, except in the case of extremists, there is not so much difference of opinion among appraisers who are well informed in the matter as would at first appear.

As stated in the paper:

"The function of valuing a public utility property consists essentially of two distinct operations:

First.—The collection of all facts and evidence tending to throw light on the question of value."

It cannot be disputed that the best evidence as to the cost of the many items composing the properties in question is to be found in the books of the company. It is not denied that the conditions of

Mr. Burgess. construction may be different to-day from what they were when the plant was constructed. Nevertheless, the actual past costs, especially of particular or individual component parts of the plant, are facts, and are always of interest and value to Courts and appraisers. Such facts may very well be considered by appraisers, even in developing evidence or opinions along the lines of reproduction new, as of to-day. In this connection, it is necessary to consider what are the apparent differences between the conditions under which construction must take place to-day, as compared with those under which construction actually took place in the past; also, how such conditions affect costs.

As a matter of fact, it seems to the writer that the cost of reproduction new method of valuing properties is largely, if not wholly, a comparative method. There is, of course, no intention of actually constructing new properties exactly like those in question, but the appraiser must consider all evidence which may throw light on the probable sum that would be required to construct such a new comparative plant. A determination of the cost of reproduction new is essentially a matter of detail, comprising determinations of the probable cost of reproducing new many items, or component parts, of the comparative plant. Although it is true that comparative costs of other properties or units and estimated costs new of the comparative plant by contractors or by engineers are all valuable aids to the appraiser in arriving at the probable cost of reproduction, it seems to the writer that unquestionably the actual past costs of the properties or units in question, as shown on the books of the company, are the best evidence for the appraiser by which to estimate the cost of reproduction new, provided, of course, that he considers and allows for all comparative conditions of construction between the past and the present.

Doubtless this is why appraisers, in many instances, have considered the past costs of new properties to be the best evidence as to their present value, because, of course, it is recognized that, in comparatively new properties, the conditions of construction under which the so-called comparative plant may be built are similar to those under which the properties in question actually were constructed. With older properties, however, this is not generally true, and it is on the older properties that there will be greater differences of opinion as to present value on account of this difference between the past cost method and that of reproduction new.

There are, however, certain very material differences in the appraisal values which may be obtained by using the past cost and the cost new as of to-day. These apply both to tangible and intangible costs.

Among the costs of tangible property included in reproduction and not in past costs are such items as the costs of cutting through

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pavements over mains and services—pavements which were constructed subsequent to the laying of the pipes. Such costs do not appear on the books of the company, but, if we are to reproduce the existing properties as of to-day and under present conditions, the costs of cutting through existing pavements is an element of the cost of reproducing the properties as of to-day. In the same way, in estimating preliminary costs, such as engineering, superintendence, interest during construction, and others, there will also be a difference between the amounts estimated for the cost of reproducing the properties and that actually incurred during the construction of the existing plant. Preliminary costs which may be entailed by the construction new of the comparative plant are purely matters of opinion based on facts and the knowledge of the appraiser as to such comparative costs of similar properties. They are estimated as an aliquot part of the complete plant as of to-day. The actual preliminary costs incident to the construction of the existing properties appear on the books of the company, and, of course, are a similar part, or proportion, but of a much less valuable property. Consequently, preliminary costs based on past costs almost invariably will be less than preliminary costs as estimated for the construction new of the comparative plant. This is because such overhead costs as engineering and general supervision frequently are paid for entirely out of earnings during recent years and do not appear in the plant account.

In the same way, there is frequently a marked difference between the results obtained by considering the two lines of evidence in determining intangible values, such as going value. The past cost method requires consideration of the actual revenues and operating expenses entailed by the plant during its early history. By some appraisers, this method is carried further to a consideration of whether or not early deficits have been compensated by subsequent earnings of the properties. By the comparative method, however, it is necessary to consider the earnings and expenses of the existing plant during recent years and to project these into the future with a view of determining the probable receipts and expenditures of the comparative plant, the construction of which is conceived to begin on the date of making the appraisal. Here again we have two lines of evidence, the one based on past costs, the other on the future, or present cost of developing the business of the so-called comparative plant. It is only an accident if they agree as to the result. Of course, going value, under the comparative or reproduction method, never can be negative, or compensated. So long as the properties are earning they have a going value.

On account of the many past appreciations and depreciations of the several component parts comprising the properties, none of which changes in value appears on the books of the company, but all of which do enter into the cost of reproduction new as of to-day, it is apparent

Mr. Burgess. that, in most instances, there will be considerable differences in the net results as obtained by using past costs and costs of reproduction new in estimating the value of utility properties, especially in the case of utilities which have been rendering service for comparatively long periods of time. As previously stated, however, it is the writer's belief that there need not be very great differences of opinion among appraisers as to the net results obtained in valuing a particular property, especially if we recognize the fact that the cost of reproduction new, though a most valuable line of evidence, is not the only line to be considered. That it is not the only line of evidence to be considered is well indicated by the Supreme Court of the United States in the Minnesota Rate Case, as follows:

"The cost of reproduction method is of service in ascertaining the present value of the plant, when it is reasonably applied and when the cost of reproducing the property may be ascertained with a proper degree of certainty. But it does not justify the acceptance of results which depend upon mere conjecture. It is fundamental that the judicial power to declare legislative action invalid upon constitutional grounds is to be exercised only in clear cases. The constitutional invalidity must be manifest, and if it rests upon disputed questions of fact the invalidating facts must be proved. And this is true of asserted value as of other facts."

In the writer's opinion, it is difficult to see how any appraiser can have at hand more relevant facts than the figures which appear on the books of the company as indicating actual past costs, especially of new properties. It must be considered that, in any particular instance, the only additional facts available are the costs of other properties generally similar in character to those in question, but, of course, not identical with them. It is the writer's belief, therefore, that, after all, there is not so very much difference of opinion among well informed appraisers as would at first appear. The cost of reproduction new is of very great value, but is not by any means the only line of evidence to be considered in arriving at the present value of properties devoted to public use. Past costs always will be considered, both of themselves, and also in determining the cost of reproduction new of the comparative plant, the conceptional construction of which is necessary in determining reproduction costs as of to-day. It must be admitted that the reproduction new method involves largely questions of opinion which are evolved from the consideration of all relevant facts. The past cost method, however, is purely a question of fact. That is the fundamental difference between the two methods.

The writer does not wish to be misunderstood as advocating the promiscuous intermingling of past and present costs in arriving at the fair present value of a property. This is, perhaps, the most frequent error that is to be found in utility appraisements. He simply wishes to emphasize the value of past costs, especially of the compo-

Mr. Burgess.

nent parts of the properties, as the best evidence which may be obtained to determine the present costs of reproduction new. From past costs of units, where available, one may very well reason to present costs. Without such past costs, all evidence of present cost is purely comparative, and, consequently, is a matter largely of opinion. Therefore, it is open to more or less serious objection or criticism, in so far as it approaches the realm of conjecture.

In conclusion, the writer believes that engineers must accept and adopt the position, so well argued by Mr. Alvord, that cost of reproduction new, as of to-day, is a principal line of evidence to be investigated in all appraisals of properties devoted to public use. He wishes to emphasize the fact, however, that this is not the only line of evidence to be considered, and that past costs, especially of recently constructed properties, afford the best evidence as to the cost new of many items which enter into the estimated cost of reproduction, as of to-day. Past costs of entire properties generally are not available, and, where they are available, may be very misleading if used to determine "the fair present value of the property used and useful to the public".

Mr. Newell.

J. P. Newell,* M. Am. Soc. C. E. (by letter.)—In this admirable paper, Mr. Alvord has failed to observe an important distinction. The Courts generally have assumed that the principles which have been established for valuation in condemnation cases are also applicable to rate regulation. In fact, there is a wide difference.

In condemnation, the owner is to be deprived of property producing present or prospective income. The duty of the appraisers is to fix the value of the property in accordance with the income.

Under regulation, the duty of the public service commission is to determine, not what the value of the property is, but what it ought to be. The value of a property devoted to the production of an income can be measured only by the income which it produces; consequently, when the commission decides that the property of a utility is worth a certain sum, and establishes rates which will bring a fair return on that amount, it has thereby established the value of the property. It has not found that the property is worth that sum, it has made it worth it. It will remain at that value as long as the conditions are unchanged. The determination of value for rate regulation, therefore, is not the ascertaining of an existing fact, but the making of a judicial decision. The duty of the appraiser is not to work out a set of mathematical calculations which will give the desired result, but to place before the commission the facts on which its decision is to be based.

* Portland, Ore.

Mr. Newell. A clear recognition of this principle will aid in avoiding some of the errors pointed out by Mr. Alvord. Particularly is this true in the consideration of "going value" or development cost. The commission is under no obligation to capitalize past deficiencies in income, nor to allow an increased rate that will exactly offset them. It is its duty to take into account the causes of such deficits, the skill and judgment shown in the management of the property, the financial conditions prevailing in the community, and the returns earned by equally hazardous private enterprises, and fix such value, or rate of return—which amounts to the same thing—as will satisfy the reasonable expectations of the investor. In this, as in all other matters pertaining to the regulation of rates, no amount of technical or legal ability can take the place of sound business sense and experience.

The writer agrees with Mr. Alvord that, in rate regulation, we must consider original cost, cost of reproduction, depreciation, probable obsolescence, past and present income, market value of stocks and bonds, in short, everything which would be taken into account by a prudent investor in a private property. He is nevertheless of the opinion that the factor which should usually have the greatest weight is the amount wisely invested, modified by subsequent changes in the prices of land, labor, and materials; in other words, the reproduction cost by the historical method.

Mr. Harte. CHARLES RUFUS HARTE,* M. AM. SOC. C. E.—In this paper Mr. Alvord makes a notable contribution to the growing literature on valuation, and one particularly commendable for its breadth of view and freedom from bias.

That there are some who will take issue with the author is made evident by the paper, entitled "The Valuation of Public Utility Property", by J. H. Gandolfo, Assoc. M. Am. Soc. C. E., but the majority of engineers engaged on or interested in valuation work can find little to which exception may be taken.

Considering these minor points, it seems regrettable that the list of definitions was not further extended; no small part of the confusion on the subject is directly due to the use, by different writers, of the same expression with different meanings.

Of those given, that of "Depreciation" is apparently a slip of the pen. It should be "A loss in value", etc., the "lessened" value, as it appears, being actually the complement of depreciation.

It is true that in certain Court decisions "value" has been defined as "measured in money", but this is not necessarily true, and the first phrase of this definition would be more strictly correct if the last three words were stricken out.

* New Haven, Conn.

So, too, in the definition of "Amortization", the last four words restrict too closely. Amortization—literally, prevention of death—is applicable to operating death of any sort, whether due to obsolescence, inadequacy, or what not. Mr. Harte.

As exactness and clearness in definitions are of the utmost importance, the use of the expression "money or its equivalent" in defining "Past Cost", and of "capital or its equivalent" in defining "Investment", intimates a fine distinction between the two, which seems hardly warranted.

In recognizing the "law as the foundation", there is a peculiar condition which should not be overlooked. The Constitution sets forth the principles on which is founded our national existence; the interpretation of the Constitution lies with the Courts; and the adherence of the Courts to precedents in a measure prevents review, unless material error can be clearly established.

There is, however, another and even more effective method of securing what is practically a reversal of a Supreme Court decision. The Constitution fundamentally is the expression of the beliefs and attitude of the Nation; if there are changes in those beliefs, or if a change is necessary in the instrument to secure proper expression of the beliefs or attitude, not only is it possible to effect such change, but seventeen Amendments testify to actual modifications; and not the least interesting feature is that the protection against confiscation of property without due compensation, now enjoyed, was not an expressed element of the Constitution as originally adopted, but is secured through two such Amendments, the Fifth and the Fourteenth; so that particular necessity exists for education as to fundamental principles, not merely as regards engineers engaged on or interested in valuation, but especially as regards the people as a whole. The mechanism by which the Constitution is amended is complicated and slow in operation, to ensure that sudden whim or passion shall not effect mischief, but it is entirely within the bounds of possibility that a change in public sentiment might result in another modification, whereby "property" would be defined so as to withdraw public utilities in part at least from the protection now granted by the Fifth and Fourteenth Amendments.

On page 150, in enunciating the Seventh Principle, the author speaks of "computed depreciation". For accounting purposes, or to bring figures found as of different dates to a common time, it is often necessary to compute extensions based on determined facts, but unless the fundamental data are determined in the field for the specific case, and the time interval is reasonably short, "computed depreciation" is, at least, as likely as not to be incurred.

Mr. Harte. There recently came to the speaker's attention the case of a property which in 1908 was reported, by a well-known engineer, to have roundly a cost to reproduce new of \$40 000, with a depreciation of \$25 000. A year later \$20 000 was put in, in additions, and \$5 000 in deferred maintenance, which apparently meant \$60 000 reproduction new and \$40 000 "less depreciation", yet five years later, with practically no maintenance, the Public Service Commission, after a field investigation, concurred with the speaker in a "cost less depreciation" of \$50 000 instead of the \$30 000 which would have resulted from following the theoretical—and, as it proved on test, inaccurate—computation of the first valuation. Although this was an extreme case, a very appreciable difference will almost always be found between the depreciation as computed and the actual conditions.

In the illustration of the appraised horse, the unresolved doubt as to the kind obtained sets forth exactly the predicament of the valuer who has used "Life Tables"; he has secured an average value, but, in order to compare it with the actual utility, it must be given the specific details that will establish its position.

On page 159 there is a statement—that "capital has properly treated the public"—which the author can hardly mean without exceptions. There have been a great many cases where capital has properly treated the public, but the unhappy tendency to restrictive and harassing legislation, which, fortunately for all, apparently is now being succeeded by a more rational attitude on the part of the public, is largely, if not entirely, due to the "public be damned" policy of some utilities.

The attitude of the utility to the public is an element of its value, however, for a condition of ill will means sooner or later unusual expenditure, either to overcome it or, less wisely, to secure necessary or desirable privileges in spite of it, precisely as good will makes for the contrary.

These comments, however, in the main touch only minor details; as a whole, the paper is admirable.

Mr. Humphreys. ALEXANDER C. HUMPHREYS,* M. AM. SOC. C. E.—Referring to Mr. Dow's remarks† as to whether we can determine the cost of a property by going to the books: Many seem to think that any information which one has a right to ask for can be obtained from the books of account. Nothing is farther from the facts, for the systems of classification vary greatly, even in the same set of books, from year to year; and in the past it was in many cases considered conservative and highly proper to charge the capital account with parts

* New York City.

† Referring to the discussion by Alex. Dow, M. Am. Soc. C. E., on the paper "The Valuation of Public Utility Property", by J. H. Gandolfo, Assoc. M. Am. Soc. C. E.

of the operating cost. The result is that many properties are now represented on the books of account by amounts which are below their cost. Mr. Humphreys.

The speaker finds much in Mr. Alvord's paper which he would be glad to discuss in detail, but will confine his remarks almost exclusively to that much-discussed and much-misunderstood subject of "depreciation", so-called.

Before taking up this subject, he wishes to enter a protest regarding the position taken by Mr. Alvord. On page 121 is found the heading, "Law—The Foundation". If it is the foundation, then we must understand what is the law. We naturally turn to the lawyers and the judges for the required information; but we find that they are far from being in agreement as to the law in respect to the valuation of railroads and other public utilities. The speaker agrees with Mr. Alvord that a "knowledge of the interpretation of the law by the Courts in their decisions and opinions" is of great importance to all. He also agrees with him that the Courts "as well as economists and engineers, must accept fundamental law as it stands; their duty is to tell us what this fundamental law means".

Unfortunately, however, the judges and commissioners (for the latter do a large amount of interpreting also) are not infallible. The result is that there is no such positive foundation for us to rest on as Mr. Alvord seems to suggest.

These men are human, like the men of other professions, and frequently they are far from being judicial in their consideration of the questions submitted to them. We must not forget that the judges are lawyers by profession, and that the lawyer, by training, is an advocate.

Our difficulties are greatly complicated because of the opinion which prevails generally throughout the country that our conditions can be improved by the enactment of a multiplicity of additional laws, rather than by the enforcement of the laws now on the statute books.

Senator Elihu Root, certainly a man who can speak on this subject with authority, recently made the following statements in a public address:

"We make too many laws. Our National and State Legislatures passed 62 014 statutes during the five years from 1909 to 1913, inclusive.

"During the same five years 65 379 decisions of the National and State Courts of last resort were reported in 630 volumes. Of these statutes, 2 013 were passed by the National Congress; and of these decisions, 1 061 were rendered by the Supreme Court of the United States.

"Many of these statutes are drawn inartistically, carelessly, ignorantly.

"Their terms are so vague, uncertain, doubtful, that they breed litigation inevitably."

Mr. Humphreys.

Under these circumstances, should we meekly submit to Court opinions, particularly when we remember how conflicting those opinions are on any one important question.

We have to submit to the highest Court when it rules, but that by no means implies that we must give up our efforts to present the truth as we see it; and on not a few questions we engineers are better qualified to present the truth than are the lawyers. This has been publicly acknowledged recently by Judge Prouty, formerly Chairman of the Interstate Commerce Commission, and now Chairman of the Board appointed by the Government to supervise the valuation of our railroads.

As the speaker analyzes the present unfortunate conditions in the United States, including placing in the hands of public service commissions authority in the three functions of government, which we, in another breath, declare must be kept separate, the speaker sincerely believes that much of our trouble has come from the almost servile attitude of the representatives of vested interests toward the men who are placed over us. If we continue in this attitude we deserve to lose our freedom and to become the slaves of a bureaucratic government. We have too much law talking and too little real law doing.

Mr. Alvord has attempted to simplify his discussion by laying down certain definitions of the terms he uses in his paper. Much of our talking at cross-purposes undoubtedly comes from the loose and incorrect use of terms. The speaker must emphatically dissent from Mr. Alvord in his definition of "reproduction cost", as he cannot see anything other than a begging of the question when the author says of this term, in parentheses, "Used in this paper generally as a short term for reproduction less depreciation". Here, through a definition of his own making, the author attempts to dispose of what is probably the most difficult question involved in the valuation problem.

When it is claimed that, in valuing plant for rate-making purposes, "Depreciation" should be deducted from the cost new of plant, what is meant by the term "Depreciation"?

If we may judge by the testimony, papers, and discussions involving this question, there is great confusion of thought as to the meaning of this term often so improperly used; but, if we follow to the end the statements and arguments of those who favor this deduction, we find that this term, "Depreciation", must be taken to cover, in connection with rate-making valuations, the accrued and to accrue liability for the future cost of renewing and replacing certain parts of plant which are to be retired from service due to any cause—physical decay, obsolescence, or inadequacy—these parts to be retired just as soon as they are no longer able to render effective and economical service, and not until then. These renewal and replacement costs are to be independent of the current repairs and minor renewals made

year by year, which are necessary to keep the plant up to the required efficiency pending these deferred major renewals and replacements. Mr. Humphreys.

It follows that such parts as are to be included in the estimate of accruing liability for renewals and replacements are those of longer life, and are not covered by the ordinary annual expenditures for upkeep; hence the estimate of annual charge against "Reserve" to meet this liability depends on the assumed effective life of the several parts and the present age as found.

We cannot fairly expect that such an estimate can be absolutely accurate. We cannot safely use, in our assumptions as to life, the data to be obtained from textbooks and the like. These tables of life expectation are supposed to be made up of averages from actual experience. If so, they certainly cannot fairly be applied in specific cases. The variations which are included to produce the average will, of course, be found also in the specific cases to which these average figures are applied; but, too frequently, these tables are only the expression of opinion of some "expert", competent or otherwise.

In any case, it is indefensible to use the life of even a certain standard piece of machinery or apparatus and apply that life to the case of another piece of exactly the same class, and even from the same manufacturer; for those of us who are familiar with operating conditions know full well how the differences in the management and upkeep of plant affect its length of useful service.

A still more reprehensible method, followed by some of those who use stated life figures as if they were proper for general application, is to quote from the report or testimony in a specific case, which figures are shown by the context to be adjusted to the specific case by considering the local and other controlling conditions. The speaker has such cases in mind, some of them where the fault has been due to ignorance or inexperience, but some of them, he is sorry to say, due to improper motives. In the most flagrant case known to the speaker, the testimony accompanying the table, not only repeatedly states that the life table is adjusted to the particular case, but the facts stated demonstrate that to use this life table for general application would be indefensible.

One feature in this table is particularly noticeable, namely, the non-inclusion of some parts having a total serviceable life of about 10 years, and this because, under the local conditions found, these parts were being renewed each year to the amount, in dollars, equal to about one-tenth of the total investment in these parts. Under other conditions, this item might carry the heaviest percentage for renewal of all the items in the table.

This exemplifies a point which has led to much talking at cross-purposes. If the renewals and replacements of all kinds fall in for renewal or replacement so that the cost thereof is found to be fairly

Mr. Humphreys. uniform, and if, by expert consideration of all the facts, it is believed that each year is thus bearing its share of the total maintenance cost of the plant as a whole, then there is no need and no warrant for continuing an annual allowance to the depreciation reserve for periodic and final renewals or replacements.

This is a condition which may develop as years go on in the operation of the property. If the total annual expenditures for renewals and replacements do become practically uniform, from then on no annual contribution to the "depreciation" reserve need be set aside for renewals and replacements. However, we still have the liability for the period prior to the time when this condition of uniform annual renewal expenditures developed. If the contributions to the reserve had been correctly estimated and computed on the sinking-fund basis, then the interest on this reserve balance would be continually required for its portion of the annual renewals and replacements. Here it is to be borne in mind that in practice there cannot be, in the case of a growing property, a complete cycle. A complete cycle would require that it be measured by the part of plant of longest life, which must be a multiple of the lives of all the other parts. In practice, even if the plant were not growing, there could hardly be any such condition; and as a result, at the end of the longest life, there would be an overlap of liability for those parts having lives which were not factors of the longest life. Then, in this case, there will be, at the end of the so-called cycle, a liability for the renewals shown by this overlap, and there should be in the reserve an amount equal to this overlap of liability. The interest on this balance to the credit of the reserve would be part of the contribution to the cost of annual renewals. In such a case, this balance in the reserve account is necessary.

This condition of uniformity of annual expenditure may well be the case with large railroad systems composed of units built at different times and spread over wide areas involving variation in operating conditions.

The same may be true of other large public utilities of varied character. As to certain items, it may be true, even in the case of smaller and closely located plants, as, for instance, the consumers' meters of a gas company, which are found to be tested, repaired, or condemned in accordance with a scheme which calls for a complete cycle in, say, 5 years.

Here this item of plant is being completely maintained by current expenditures, and hence there is no additional liability to charge to profit and loss and to credit to reserve.

Now, the speaker's contention is that if the plant is maintained so as to render to the public thoroughly effective and economical service, there is no just reason why a deduction should be made because the time has not yet arrived for the final renewal or replacement of certain

of the parts, for the renewal of which the corporation is liable and will be held liable by the public service commission. Whether the plant as a whole is maintained by current annual expenditures alone, or by the current ordinary expenditures plus the expenditures for deferred renewals covered by a reserve maintained by annual contributions, does not affect this question of deduction from plant value for depreciation. If the plant is maintained as well as it is possible to maintain it, and if, as it ages in certain of its parts, it is made young again by the renewal of those parts, then the owners are doing all in their power to preserve their property, in the interest of themselves and the public. Under these circumstances, to deduct for the accrued liability for renewals—a liability which cannot be met until the time for economical renewal arrives—must be confiscatory. Mr. Humphreys.

Further, the speaker believes that the day will come when the U. S. Supreme Court will so decide. It certainly will do so when at last its members come to realize that the basis for "depreciation" deduction is the acknowledgment on the part of the utilities of their liability for periodic and final renewals, which acknowledgment does or should take the form of certain statements on their books of account which act to prevent over-statement of profits and the payment of dividends not earned, and also places the burden of renewals on the consumers benefited, instead of on their successors. A reserve carried for periodic and final renewals of plant is simply an accounting device in the interest of more accurate statements of earnings and better financial practices.

Finally, what the speaker is contending for is that, in the case of a property honestly and economically managed, including the maintenance of the plant at a high degree of efficiency, we should have the privilege, when necessary, of spreading the cost of deferred plant renewals and replacements over all the probable years of service of each part of the plant by means of certain accounting processes, so that we shall not deceive ourselves as to our annual profits and shall place the current burden for upkeep where it belongs; and also we should enjoy this privilege without affording an excuse to commissions and Courts to deduct this reserve accumulation from cost new of plant by reason of the supposed acknowledgment on our part of actual plant depreciation.

H. F. DUNHAM,* M. AM. SOC. C. E. (by letter).—With an able attorney, the writer once examined a simple machine that showed the effect of the direct expansion of a steel rod. There was no leverage, nor multiplication of parts, nor of motion. A weight was held in place by friction. If the rod expanded, the friction was lessened and the weight fell. If one merely put his fingers on the rod, the weight Mr. Dunham.

* New York City.

Mr. Dunham.

went down. If he breathed once upon it, the same result followed. A slight force applied to the end of the heavy supporting cast-iron bed-plate warped the casting, and the weight descended.

The attorney said:

"This is the finest object lesson of a life time. Now it is known there can be nothing firm or rigid on our planet. Everything is flexible. The mountain ranges move away from the sun. Nothing is fixed."

At that moment, laws and Court decisions would have seemed to that attorney less permanent than they appear to be to the author of this illuminating and interesting paper. His masterly way of meeting the difficulties everywhere prevalent, and so well set out, will not suffer if attention be drawn to this one feature of assigning to Court decisions an unexpected degree of permanence. The paper itself is not only a protest, but, through its merits, may do much toward a change in legislation or opinions about which the author has taken a hopeless view.

In the discussion of another paper,* the writer referred to new rules from the Courts, holding that the rules are valuable, but should not be substitutes for vital individual work no less searching than that which they embody.

After all that has been written and said about methods and details in the determination of rates, there may be little patience with or space for new suggestions; and yet it seems that for each particular city there should be an appropriate valuation and a schedule depending on, not only all the local conditions of cost, construction, efficiency, permanence, etc., but on other known facts pertaining to that city, surrounding cities, and even the relation of these to the State. A schedule thus determined should be a schedule for that city, and should vary but little, whether the ownership were private or municipal. To show how easily an appraiser is led beyond the municipal boundary, it is only necessary to cite cases that are quite common in many parts of the country.

A city is located and established in such a way, with respect to any possible water supply, that great expense for installation and operation must be incurred. A valuation is based on these conditions. The rates naturally and comparatively would be high. In a near-by city quite opposite conditions exist. Excellent water can be obtained and purveyed with small expense and attention. Naturally, the schedule would be a low one. Resting each case on the same principles, and fixing valuation and rates in the same manner, it may be easily shown that one city would be supplied with water at a fraction of the cost in the other. The question to be met by appraisers,

* *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 373.

and by utility commissions as well, is whether other features than those pertaining definitely to the works in those cities should be considered. Where there is a little rivalry for manufacturing and other lines of development, it would be extremely difficult to prevent the common council from making sacrifices in some way through which their rates for water would be brought down to the level of those in the more favored community. Added weight is given to this view by the fact that, if the plant were owned by a private company, the same logic and necessity would have like weight and influence. There is no doubt that some modifications would follow, and the fact that this result would be reached in a business way by any group of business men furnishes sufficient evidence that attention must be given to the principles that lead to such action. The existence of the conditions in the second city would affect the valuation of the plant in the first city. These principles have their place, in common with any of the other facts on which valuation and rates may be based. That a commission would recognize them as important and legitimate can readily be inferred. Their action in the matter would be a recognition of business principles not unlike that which obtains in fixing differential rates in transportation. A fundamental reason or guide for their conclusions would be found in the fact that the two communities were a part of the larger commonwealth, and, in fairness, the best interests of the whole should not be neglected. Mr. Dunham.

The relation of a commission to natural conditions that affect unequally two near-by communities can be illustrated by assuming that through some oversight the rates for the city that could obtain a supply only at great expense were based on a 7% return, while 5% had been fixed for the more fortunate city. This would be followed by protests of greater force than could be expected, if the percentages of return had been reversed, further showing the existence of substantial reasons for introducing a wide equalization method. It might be difficult to say to what extent a commission has the right to introduce differential or compensating rates in adjacent cities, but the right should be better sustained than is the right of a commission to fix meter rates for water at a high figure in one city and at figures below the cost of pumping in a similar city obtaining its supply from similar sources and under like construction expense.

The refusal in some States to permit further bestowal of free water can only prove beneficial to all, but it should not be forgotten that to introduce rates below actual cost is not unlike a return to free water privileges against which the same authority enters protest.

The responsibility of a city for its own part in the design and construction of a company utility is not often referred to, but this affects valuation. Mistakes at the beginning, or at a later period, usually involve both city and company. The city's approval is required

Mr. Dunham. in connection with the design and on the completion of the works. Extensions are usually defined and ordered by the city. If it becomes apparent that errors have been made and funds expended that fail to bring proper returns, it can hardly be claimed that only one of the contracting parties should suffer. In a recent case, a city ordered an expensive intake pipe from a river for fire protection; then a city sewer system was constructed in such a way that the intake pipe could not be used; then, under a purchase clause, the city obtained the plant, demanding that the cost of the intake pipe should not be included, as it had no present value. This demand was refused by the State commission. A growing spirit of fairness on both sides lessens many of the difficulties in utility affairs.

Mr. Knox. STUART K. KNOX,* M. AM. SOC. C. E. (by letter).—The one ground for really serious difference of opinion in Mr. Alvord's otherwise admirable review of the Fundamental Principles of Public Utility Valuation, will be found by most engineers in the author's treatment of the subject of depreciation in its relation to the value on which rates should be adjusted to yield a fair return.

The writer infers from this and other recent writings by the same author that Mr. Alvord is still of the opinion that the deduction of physical depreciation from value new in rate-making appraisals is morally justifiable and legally necessary, even when the so-called annual depreciation reserves are computed on the sinking-fund basis.

This idea, still widely held, appears to be based largely on two assumptions, both false, as follows:

(*a*) That the deduction is morally justifiable because appreciations, in the form of unearned increments, are, perforce, credited to the owner.

(*b*) That the deduction is legally necessary because of the existence of some fundamental principle of law which has compelled the Court so to decide.

Thus Mr. Alvord, in closing the discussion of his former paper, "The Depreciation of Public Utility Properties as Affecting Their Valuation and Fair Return,"† says:

"Objectors [to the deduction of physical depreciation where reserves are computed on the sinking-fund basis] do not take note of the fact that the appreciations of plants and property are included by the reproduction method, and must be considered in connection with depreciations and that, if the objectors would exclude depreciation, they must logically exclude the appreciation that the reproduction

* Montclair, N. J.

† *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 788.

method usually introduces. The same arguments which one would use to exclude depreciation can be profitably used to show that questions of appreciation should not be entertained." Mr. Knox.

Thus, also, in the present paper, he lays stress on the fact that the Supreme Court of the United States, under compulsion of the Constitution, which secures an owner in the possession of unearned increments, has held: "that the value of the property is to be determined as of the time when the inquiry is made regarding the rates", and clearly implies his (erroneous) belief that the deduction of physical depreciation is in all cases necessary in ascertaining the value of a property "now".

The assumptions, (*a*) and (*b*), having been characterized as false, it remains to show wherein and to what extent they are so.

Assumption (*a*) is false because physical depreciation is not the antithesis of any appreciation which should be allowed. There is no method of valuation now in use which credits an owner with appreciations in value without automatically debiting him with their antithetic depreciations. Thus the antithesis of appreciation in the value of real estate is depreciation in its value; and the application of the reproduction cost method, which credits the owner with one, debits him with the other. The same is true of appreciations in the value of labor and materials, which may have occurred since the date of original construction, of appreciation due to the placing of pavement over pipe, and of other increments which are the result of what might be called external causes. Each has its antithetic depreciation, and the method which allows for one takes account of the other. Not even the advocates of original cost to date as the proper basis for rate-making advise depriving the owner of unearned increments without, at the same time, saving him harmless from unmerited decrements in the value of his property.

The foregoing facts were recently emphasized, as the writer now recalls, by George F. Swain, Past-President, Am. Soc. C. E., who pointed out that a sharp distinction should be drawn between physical depreciation resulting from use and the shrinkage in value which may be caused by recessions in the market value of real estate, the discovery or invention of cheaper and more efficient methods of manufacture and construction, and the like.

The wide-spread failure to perceive, or inability to realize the important and real differences between these fundamentally dissimilar things appears to be due in no small measure to the indiscriminate and unfortunate use of the single word "depreciation" to designate both of them. (See Mr. Alvord's table on page 131.) "Deferred renewals", "Deferred replacements", or some similar designation

Mr. Knox. would describe more aptly and accurately what is meant when physical depreciation is spoken of, than does the word "depreciation".

The term "deferred renewals", thus used, for example, might be defined as Mr. Alvord has defined "depreciation", page 120, as "the lessened [decrease in] value of any property, structure, or machine, due either to its wear, loss of usefulness, growing lack of adaptation, or approaching abandonment", or it might be defined otherwise, as "that which gives rise to the necessity for eventual replacement", or as "wastage or consumption of physical plant through use, to offset which so-called depreciation reserves are permitted to be made from earnings, before computing net income".

"Deferred renewals" does not connote the antithesis of an unearned increment. It does connote that steady and inevitable wearing out of plant and equipment, the entire cost of which is as much a part of operating expense as is the cost of coal. It connotes, also, the close analogy which exists between the thing described and current repairs and renewals, from which it differs materially only in respect of the magnitude, and evenness in annual distribution of the replacements required, and with which it merges completely when this distribution is naturally uniform.

Every intelligent investor who places money in public utility or other property, does so with the full understanding that it will be subject to the normal vicissitudes which beset invested capital everywhere. The investor expects to be, and is, debited with all losses resulting from bad judgment, miscalculation of future growth, and the various unforeseen and unforeseeable contingencies which cause the value of invested capital to fluctuate. No one, for example, ever heard of a fund being amortized out of earnings to offset a shrinkage in value due to the bursting of a real estate boom. In consideration of the assumption of these risks, every investor expects to reap the benefit of his own good judgment and foresight. He rightly expects to be credited even with unforeseen and unearned increments with which fortune may favor him, and these increments in value are in fact vested in him by the Constitution of the United States, as interpreted by the Supreme Court.

Physical depreciation is not such a vicissitude as is above described. It is neither unforeseeable nor unforeseen, but is an operating cost the occurrence of which is as certain, and the amount of which may be predicted with as much confidence, as future taxes. This principle has been fully recognized by the Courts, including the Supreme Court, and this fact brings us to the consideration of what the attitude of the Courts actually has been with regard to this matter of depreciation in its relation to value, and to the examination of Assumption (*b*), characterized as false, "That the deduction is legally necessary

because of the existence of some fundamental principle of law which has compelled the Courts so to decide". Mr. Knox.

The writer does not recall an instance in which the Courts have decided with specific reference to the situation where reserves have been, and will continue to be, computed on the sinking-fund basis, that physical depreciation must be deducted from value new in rate-making appraisals. Certainly no such ruling is discernible in any of the decisions quoted by Mr. Alvord. The reader searches these quotations in vain for any reference to the method to be used in computing the annual reserve, whether sinking fund or otherwise, and this is the crux of the whole matter.

So far as the writer knows, it is not seriously contended by any valuation expert that the owner will suffer financial loss, if, where reserves are computed by the straight-line method, physical depreciation is deducted from value new in ascertaining the value on which the owner should be allowed to earn a fair return. As was demonstrated at length in the discussion of Mr. Alvord's previous paper, the value on which the owner should be allowed to earn a fair return depends wholly on the provisions made to offset physical depreciation, and on whether or not these provisions effect a virtual refund of capital. If they do effect a refund of capital, then it is equitable, in rate-making appraisals, to deduct from value new the amount of the capital thus refunded. If the amount thus refunded coincides with the physical depreciation, it is equitable to deduct the physical depreciation, and this is true whether or not the refunded capital is kept in bank as an actual tangible depreciation fund.

The writer has stated that he recalls no instance in which the Courts have decided with specific reference to the situation where reserves have been, and will continue to be, computed on the sinking-fund basis, that physical depreciation must be deducted from value new in rate-making appraisals, and it is certain that the Courts could not thus decide without involving themselves in a fatal inconsistency.

The Supreme Court of the United States, in the Knoxville case, ruled that:

"The Company is not bound to see its property gradually waste without making provision out of earnings for its replacement. It is entitled to see that from earnings the value of the property involved is kept unimpaired, so that, at the end of any given term of years, the investment [so far as the effects of this wasting is concerned] remains as it was at the beginning."

If this means anything of value to the owner, it means that the Courts will not suffer him, against his will, to be subjected to loss solely as a result of the inevitable physical depreciation of his plant.

Mr. Knox.

Now it is a fact, susceptible of mathematical proof, that an owner whose annual reserves are sufficient only to amortize the sums required for replacement, when credited with interest accretions, *i. e.*, whose annual reserves are correctly computed on the sinking-fund basis, does suffer a loss from no other cause than the wasting of his property through use, unless rates are adjusted to yield, in addition to operating expenses and this annual reserve, a fair return on "value new". For if his property suffers no physical depreciation, his profits represent a fair return on "value new"; whereas, if it does suffer physical depreciation, his profits represent a fair return on an amount less than "value new".

Therefore, under these circumstances, if a Court rules as the Supreme Court did in the Knoxville case, and at the same time holds that rates must be adjusted to yield a fair return only on value new less physical depreciation, it is obvious that it has decided two interrelated and interdependent matters in inconsistent and irreconcilable ways, and that the question confronting the engineer, in the premises, is not whether he shall cut his valuation to fit Court rulings, but by which of two conflicting decisions he shall be governed. The presumption is strong that the Courts have not, either knowingly or unintentionally, placed themselves in so anomalous and untenable a position.

In Mr. Alvord's language, the thing which is valued is a property. The Courts have said that it is the property which is used and useful to the public; but a reserve fund which is uninterruptedly used in amortizing and supplying the sums needed for necessary replacements is as much part of the used and useful property as is a pumping engine or a locomotive. Few engineers would question this if the fund were a tangible one actually maintained in bank, and as soon as one admits this fact he is committed to the proposition that any fund thus computed, regardless of its disposition, is a part of the used and useful property. For if the owner diverts the annual reserves to other uses, the fund will not be available when required for replacements, and these, therefore, must be paid for out of the owner's pocket. Eventually, he (or the stock and bondhholders) will be obliged to supply, from his own resources, not only the annual sums diverted, but also the accumulated compound interest on these amounts. The net effect of the operation, therefore, is that the company has loaned the depreciation reserves to the owner (or to its stockholders) "on call" at the rate of interest at which the fund was computed, and that the entire fund with estimated interest accretions remains part of the used and useful property as much as if it were actually in bank. There is nothing in the principle, that the used and useful property is the thing to be valued, which would act as a bar to the inclusion as value of the accrued physical depreciation.

The Courts have also said (and Mr. Alvord makes a great point of this) that it is the value of the property as of the date of the appraisal which is to be ascertained. Although there is discernible in the decisions of the lower Courts the same misapprehension which leads Mr. Alvord to confuse physical depreciation with ordinary shrinkage in value, one cannot but be impressed with the fact, in reading these decisions, that the real rock on which the Courts have taken their stand is that it is the value of the property "now" which must be ascertained. The reasoning by which the Courts have reached this conclusion is as clear as noonday, and the logic quite unassailable. The Constitution secures the owner in the possession of unearned increments. It follows logically that he should be obliged to bear losses resulting from antithetic decrements. In applying the reproduction-cost method, it is the cost of reproducing the property "as of to-day", using average current prices for labor and materials, which must be ascertained, and not the cost of reproducing the property under the conditions which existed at the time it was constructed. In deciding this point, the Courts are on accustomed ground. It is undoubtedly true that logic has compelled them to decide that the value to be ascertained is the value "now"; but the conclusion that physical depreciation must always be deducted in arriving at the value now is demonstrably a *non-sequitur*. Mr. Knox.

If it be acknowledged that a given property at a given time has an intrinsic or fair "value new", which may be ascertained independently of a consideration of rates (and the whole theory of the valuation of rate-controlled public utilities is predicated on this assumption), and if it be admitted that the accrued physical depreciation on a given property at a given time is a definite quantity, the value of which may be ascertained with reasonable accuracy, then we are committed to the proposition that "value new" and "value now" are synonymous expressions, so long as the so-called depreciation reserves are computed on the sinking-fund basis.

For, let V_N = value new, D = accrued physical depreciation, and V_P property value, or value new less physical depreciation. Then we may write the expression:

$$V_N = V_P + D,$$

in which V_N, D, and hence V_P, are definite quantities which may, by hypothesis, be ascertained independently of a consideration of rates, for any given property at a given time.

Now the "value as of the date of the appraisal" or "value now" of a property, the average life expectancy of which has been diminished as a result of use, is represented by and equal to the total capital investment which a willing purchaser must make to acquire it from a seller who is willing, but not compelled, to sell.

Mr. Knox. To acquire a property on which there exists an accrued physical depreciation, D, a purchaser, A, would be obliged, and could afford to pay to the owner, B, a sum equal to its deteriorated property value, V_P, as V_P is an intrinsic value ascertained independently of a consideration of rates, and as such must, of course, be sustained by the rates; but, if accrued physical depreciation and future depreciation reserves are to be computed on the sinking-fund basis, V_P will not represent the total investment necessary to acquire the property, and hence will not represent value now. If the purchaser, A, will be allowed to reserve annually from earnings only such sums as will with interest accretions amortize the value new of each plant element, in a time interval equal to its age plus its remaining life expectancy (and this is the correct basis for computation of a proper sinking-fund reserve), then the total investment necessary to acquire the property will be $V_P + D$. For if the purchaser, A, is to place himself in a position to make replacements as these become necessary, without the investment of new capital, he must on the date of the purchase, set aside at interest an amount, D, equal to the physical depreciation on the plant. If he does not, he has no less effectively committed himself, because, in taking over an old property in exchange for a sum, V_P, equal to its intrinsic value, he has also by hypothesis assumed the obligation at some future time to invest in replacements additional new capital having a present value of D.

Now, as things equal to the same thing are equal to each other, it follows that:

Value new $= V_P + D =$ value now, and that when depreciation reserves are computed correctly on the sinking-fund basis, "Value now" and "Value new" are synonymous expressions, which is what it was desired to prove.

Thus the Courts are correct in ruling that "value now" is the proper value to use as a basis for rates, and incorrect, if at all, only in failing to perceive or recognize that the question of whether or not physical depreciation shall be deducted from "value new" depends entirely on the provisions made to take care of future replacements, and on whether or not these provisions are such as virtually to effect a refund of the capital wasted through use.

The writer has already, in his discussion of Mr. Alvord's former paper, reasoned at length with regard to the relationship between the various methods of providing for past and future physical depreciation, on the effect of the several methods on fair rates, and on the values which serve as their basis. He will not repeat that discussion here except to say that no one of the conclusions there reached is vitiated by Mr. Alvord's just criticism that it was written primarily with reference to cases in which Value New, Reproduction Cost New,

and Total Cost to Date were synonymous expressions. "Fundamental Principles of Public Utility Valuation", the title of Mr. Alvord's present paper, must be sufficiently broad in scope to cover all cases which may arise, and cases frequently do arise in which value new practically coincides, not only with reproduction cost new, but in recently constructed properties with total cost to date, as well. Mr. Knox.

C. E. Grunsky,* M. Am. Soc. C. E. (by letter).—Although he does not agree with the author in his conclusions, which are based on the assumption that, because the Supreme Court and other Courts have reached a decision, certain fundamental principles are finally settled, the writer desires to compliment him on his attempt to establish such principles. There is need at the present time for careful consideration, and much good should come from the discussion of the question, now squarely presented: Must the opinion of the United States Supreme Court be accepted as final, even though it be an economic fallacy? Perhaps such opinion may be in conformity with the law. If economically wrong, should not the law be changed? Mr. Grunsky.

The net profits and, consequently, the value of any property, such as a public utility, other conditions remaining the same, depend on, and fluctuate with, the rates. The rates determine value, and therefore the converse, that value should be the guide in fixing rates, is fundamentally wrong. It is the height of absurdity for the law, the Courts, or public utility commissions, to make value—however defined or restricted—a basis for the fixing of rates.

The uselessness of thus moving in a circle—which follows when value is considered in establishing rates—can, when properly pointed out, be grasped by the dullest mind, and is gradually being recognized by those who are concerned with the establishment of rates.

If the law is at variance with this fundamental truth, the law should be changed. Where the interpretation of the law by the Courts is at variance with this fundamental truth, this interpretation should be modified. Engineers and economists should not accept supinely the erroneous conclusions of the Courts, but should continue the campaign of education until the true principles are recognized and firmly established.

The writer, therefore, cannot agree with the author's conclusions which are based on the premise that, because the Courts have said so, some method must be found to establish fair value of the public utility under the assumption that earnings may be regarded as dependent on, instead of as the cause of, value.

W. Kiersted,† M. Am. Soc. C. E. (by letter).—The first paragraph of this paper presents the key-note of the author's argument, namely Mr. Kiersted.

* San Francisco, Cal.
† Kansas City, Mo.

Mr. Kiersted. that "the fundamental principles involved in valuation work * * * are to be found only in the law and the interpretation of the law by the Courts." This view as expressed seems somewhat restrictive, for, on reflection, it must be conceded that the fundamental principles of valuation work and all kindred processes are the principles of equity and justice, or those principles which inspired the minds of the framers of constitutional government, which should be the mainspring of all law, and are the beacon lights guiding the Courts in the interpretation of law. Moreover, the author will agree that the interpretation of a law may be modified in the light of new and better evidence and to conform to legislative amendments.

A review of the Court's decisions, from which the author has quoted so freely, must impress one with the design of the Court to place few restrictions on efforts put forth in valuation work, so long as such efforts are inspired and controlled by motives of equity and justice. This is particularly true of the late Mr. Justice Brewer's decision in the case of the National Water Works Company *vs.* Kansas City, Missouri, 62 Fed., 853, with which all engaged in valuation work are familiar, with the exception, perhaps, of that portion of the decision not often quoted and reading as follows:

"The city by this purchase steps into possession of a water works plant, not merely a completed system for bringing water to the city and distributing it through pipes placed in the streets, but a system already earning a large income by virtue of having secured connections between the pipes in the streets and a multitude of private buildings. It steps into possession of a property which not only has the ability to earn but is in fact earning. It should pay therefor not the value of a system which might be made to earn but that of a system which does earn.

"Our effort has been to deduce from the testimony that which in this view of the situation can be safely adjudged (the fair and equitable value). The original cost of the works is not accurately and satisfactorily shown. If it would have assisted us in reaching a conclusion, if in consequence of our ignorance thereof we have not placed the value upon the property which it deserves, the company alone is to blame, for by the production of its books it could have clearly shown the actual cost of every part and of the whole of the property. There is a large amount of testimony as to the probable cost of reproducing the system, to which strenuous objection is made on the ground of an alleged temporary and extreme depression in the cost of labor and material.

"We have before us the estimate placed before us by two gentlemen of experience and capacity, appointed as commissioners, with direction to report 'the fair and equitable value', but neither by order of the court appointing them nor by their report are we advised as to what they considered the criterion of the present 'fair and equitable value.' If they added anything beyond what in their judgment was the reasonable cost of reproduction we are not advised as to how much

they added or what they took into consideration in making such addition. We have the fact of liens placed upon the property to the extent of three million dollars with the qualified approval of the city officials. Mr. Kiersted.

"We have also the statement of the earnings and the estimate of the value upon the basis of a capitalization of those earnings amounting, as stated, at 6 per cent., to four and one-half millions. Rejecting the latter as too high and the cost of reproduction as too low, and taking into consideration the entire history of the transactions between the company and the city from its commencement to the present time, we have sought to place the value upon the property as it stands, with all the connections already made between the pipes and the private and public buildings and with the work which it is in fact doing of supplying all of these buildings with water and receiving pay therefor, and that valuation, with much discussion, comparison of figures and readjustments, we have agreed is three million dollars."

After considering collectively the various Court decisions of rate cases, the question arises, is there not a tendency among engineers, in their work of valuing public utility property, to particularize and to restrict the application of the language of the Court to some theory or particular method of valuation, with the ultimate aim of formulating a rule of procedure to be followed in all such work to the discouragement of those who would follow some other method? There seems to be a tendency in this direction with regard to the reproduction method of computing cost of physical property, on the ground that this method, having received the approval of the United States Supreme Court, should therefore become an exclusive one.

It does not appear, from the opinion of the late Mr. Justice Brewer, that the theory in itself is a complete measure of either cost or value, at least to the extent and in the manner used in the Kansas City case. Other decisions which seem to place considerable emphasis on the "reproduction method" doubtless regard the application of the method in a much broader sense than do some persons actually engaged in valuation work. The author states that reproduction "has not been originated by engineers to torment the Courts, but it has been concluded by the reasoning of the Courts to be a logical and important line of evidence", and then goes on to elaborate certain "conceptual" processes regarded as necessary to a proper frame of mind preliminary to the work of valuation, and to give several illustrations, with the evident intent of broadening and extending the viewpoint of the appraiser, and finally to sum up the application of reproduction to valuation work in a manner more restricted than is to be expected in a paper dealing with the underlying principles of public utility valuation.

The reproduction theory really embraces three distinct methods of valuation:

Mr. Kiersted.

1.—Present-Day Conditions and Prices.—This is the author's method, and consists of estimating the cost of reproducing the inventoried property as a whole under present-day physical conditions at present-day prices of labor and material, or at prices of labor and material representing the average of a few preceding years. This method allows the unearned increment to be included in cost to whatever extent the prices of labor and material exceed those of the past, and the construction costs, which would be incurred under existing physical and urban conditions, exceed those actually incurred under the original physical conditions. It also depreciates indirectly the actual investment in present-day property to whatever extent the present-day prices of labor and material are below those of the past, and the present-day conditions are more favorable for economical construction than those of the past. Structural costs thus obtained are reduced by the amount of depreciation that may have accrued, on the theory that the property possesses limited life and that any method which admits elements enhancing the value of the property should consistently make allowance for those other elements serving to depreciate the value thereof.

2.—Original Conditions and Present-Day Prices.—This method, fully presented in the Report of the Special Committee on Valuation of Public Utilities, of this Society, consists of estimating the cost of reproducing the inventoried property as a whole, under the original conditions, but at present-day prices of material and labor, thereby excluding certain unearned increments, allowed in the previous case, due to change from original to present-day conditions, but including such enhancement as may result from an increase of prices of material and labor, and as may result from a change from a new-structural condition to a permanent, settled, operating condition. Cost of reproduction-new thus obtained is reduced by the amount of accrued depreciation in order to arrive at present value for rate-making after having given due consideration to development costs and similar elements of value.

3.—Original Conditions at Original Prices.—This method consists of estimating the cost of reproducing an inventoried property in a piecemeal manner corresponding with the piecemeal construction of the past, under the original physical and urban conditions at the original prices of material and labor, as nearly as they can be ascertained; and thereby includes no unearned increments. Development cost is computed by the deficit method. Discount on bonds to a reasonable extent is taken as an element of cost, unless provision is otherwise made to amortize the discount during the life of the bond as a charge to operating expense. Exception in the case of urban real estate is sometimes made in giving it a present-day value, on the theory that urban real estate possesses tangible and usable value

wholly independent of the public utility with which it is associated. This method differs essentially from the investment method, because, in taking account only of the property inventoried as useful for public utility purposes, it excludes from consideration any items of property which may have been abandoned or passed out of use for one reason or another, thereby depreciating correspondingly the investment. It anticipates, further, that in the future any item of property replaced by a new one is charged off the capital account, and the value of the substitute item is charged thereto. No deduction is made from reproduction as thus determined for accrued depreciation in a rate case, for the reason that the owner of the property should earn on a 100% value of property thus valued, and that, so long as replacements are timely, the property possesses perpetual, or, more properly speaking, unlimited life. Mr. Kiersted.

Most of the valuations of physical properties which have been made in the past are a sort of conglomerate, partaking to a greater or less degree of all three of the methods just outlined. Some recent appraisers have attempted to particularize, without any wide difference in the final result. Considered in this light, it can scarcely be conceived that any one of the three methods, logically, fairly, and consistently followed, would fail of due recognition from the Court, although a substantial agreement of the final result of the three methods, respectively, used independently, might be considered more conclusive evidence of cost or value as the case may be. Moreover, the inventory of a public utility property should be prepared and itemized so that appraisers can use any and all these three methods of valuation if sufficiently reliable data are available for the purpose, as is the case of most of the small non-competitive utilities. With most railroad properties and other large, old, and complex properties the case may be quite different.

The owners of public utilities would do well to see that the inventory of property is prepared so as to facilitate independent valuation from several points of view, and, in addition, to compile a statement of original costs year by year, as complete as possible, as an aid to the public service commission. For such a commission, in arriving at a value which will be fair and equitable to the rate-payer as well as the owner of the utility, is not likely to be influenced strongly by the view of any single expert or any commission of experts reaching a conclusion by a particular and exclusive method.

Cost to Value.—The author has laid considerable stress on the proposition that cost is not value, and that, after ascertaining the cost, and not until then, should the mind of the appraiser be directed to questions relating to value. It is naturally expected that the Courts, taking a broad and comprehensive view of the question of valuation

Mr. Kiersted. of public utilities, would discuss the relation between cost and value academically; but those engaged in the practical work of valuation should be able to supply in a specific case the connecting link in the line of reasoning between cost and value when making the distinction. It is conceivable that such a distinction may exist, but it is probable that it more strictly applies to competitive public utilities like railroads than to monopolistic utilities like water-works. The writer in the discussion of Mr. Nicholas S. Hill's paper on "Valuation of Public Utilities" read before the Society of Municipal Engineers of New York, made a distinction of this character, as indicated by the following quotation from that discussion:

"The great interstate public carriers are essential to the general development of the country as a whole, and make possible the assembling of the population in widely separated communities and the supply of those communities with all the necessities of life which surrounding territory produces and the comforts and conveniences which modern city development demands. This mutual dependence between a common carrier and its particular dependencies is also manifest in a somewhat different manner, but none the less important, between two cross country railroads which may be serving independent territories under non-competitive conditions, but which are truly competitive as to coast to coast traffic. The one railroad may pass through more non-productive country than the other, the one may be capable of a greater carrying capacity per ton-mile by reason of superior grades and alignment than the other, and the one may have superior coast terminals and may have cost much more to construct than the other.

"The cost of these two lines of road is one thing, but the value, the earning value of each, is quite another thing. One railroad may have adopted a policy of building quite as much for the future as for the present and may have a much greater investment mile for mile than its competitor which may have been built more with reference to present requirements with the expectation of improving structures, alignment and grade as its traffic increases. The one road may be said to have been overbuilt in the language of the paper under discussion, but this fact should not in any way militate against present value. In a rate case values of the carriers, which are competitive as to through traffic and non-competitive as to local traffic, should be so adjusted that each can earn a fair return though not necessarily at the same rate, and thereby be enabled to give proper service to the territory dependent upon each for business prosperity and general development."

To the statements just quoted the writer would add that, even in the case of competitive properties like railroads, there may be found less difference than is presumed to exist between cost, properly estimated, and value of the individual property, regardless of the wide variation of structural costs as between independent properties, because the earning capacity of the respective properties, being dependent on operating as well as structural conditions, may admit

of such an adjustment of competitive and non-competitive rates that each property may earn a fair return on a value based largely or wholly on reproduction cost considered in its broad sense. Mr. Kiersted.

The cost of a public utility property, fairly determined by whatever method or methods, represents a state closely approximating the facts, and whenever an appraiser departs from his belief as to fact, on the presumption that cost is not value, and attempts to reason from cost to value, he enters the realm of speculation, to a greater or less extent, and will experience difficulty in finding a well-defended or defensible landing place. In fact, there will always be at least two conceptual values, one above, and the other below, the reproduction cost, to confuse in reaching a definite result through such a line of reasoning.

Although the question of determining value possesses business aspects and requires consideration from the business man's point of view, still the determination of value for purchase and sale is separate and distinct from that of value for rate-making. In other words, commercial value, the business man's value, is not necessarily earning value, and *vice versa*. For instance, in attempting to reason from cost to value, it may be argued that reproduction new, under present-day conditions, should take into consideration the experience gained in the construction of the existing property, and, reasoning from this general proposition, occasion can readily be found, as in valuing a water-works, to substitute an equivalent single main for two or three existing mains, a new water supply from the same or a different source at a less cost than the existing water supply, new and modern pumping engines for old and non-duplicatable machines, and so on down the line for each item of the property, finally to arrive at a value of a substitute plant capable of rendering service as efficient as the existing plant at a cost considerably below that of reproducing an identical plant. Would such reasoning be just?

Take a further example of a town which, through loss of certain business resources, may have become to a degree depopulated, or that of an owner of a public utility property who, in anticipating future requirements, may have built in advance of present-day requirements. The investment in each case is a fact; but the situation that a town may have become to a degree depopulated, cannot properly be advanced as an argument for depreciating the original investment, judiciously made, any more than it can be advanced as a proper reason for an increase of rates. It is clear, however, that the rate of return on the investment must be less than it was before the town became partly depopulated. Suppose the author's views with respect to reasoning from cost to value obtained in this case, and the original investment was depreciated 50% because of the depopulated condition of the town, and a 7% return was allowed

Mr. Kiersted.

on the depreciated value, and that thereafter the business of the town revived sufficiently to restore the original population conditions without any addition having been made to the property, and in the meantime the net earnings having doubled—would a public service commission, on the one hand, be justified in cutting the rates on petition of the rate-payer, or, on the other hand, on petition of the owner, could the commission find satisfactory reason for removing the excessive depreciation formerly applied and thereby make good the actual investment, in order that the owner might receive a fair and adequate return thereon? The case of the owner of an over-built property is a parallel one.

It looks to the writer as though the author, in introducing the "cost to value" line of reasoning, is confusing the process of rate-making with that of valuation—two independent processes, each complete in itself, in natural sequence.

The fact is that a public service commission, having received or made complete estimates from several viewpoints of the cost of producing an existing property, and having compared the same with the accounts of original cost to the fullest extent possible, and finally having reached a conclusion with due allowance for interest, the cost of financing and development costs, has accomplished all that can be expected to be accomplished in arriving at a fair earning value of the property. The commission then proceeds, by an entirely independent process, to determine a schedule of rates which will yield an income duly providing for operating expense, repairs, general maintenance, replacement fund, operating contingencies, and a return to the owner, in such an amount and at such a rate as, under the particular circumstances, will be just and equitable to both rate-payer and owner. In this process of rate-making the commission, not being confined to some specific rate of return, can provide for such reward as good management and good engineering may be entitled to, and can make such discrimination between the rate of return on over-built property and under-built property about to be improved, as may be fair and equitable, and thus avoid unnecessary and phantom distinctions between cost and value. With these considerations in mind, it can scarcely be conceived necessary or advisable to exclude all thought of value during the detailed process of valuation and rate-making of a public utility.

Mr. Burns.

CLINTON S. BURNS,* M. AM. SOC. C. E. (by letter).—The writer is impressed with the broad-gauge treatment that the author has given to this subject, and wishes to add his endorsement of the fundamental principles therein presented. He wishes not only to add his endorsement, but to add emphasis to one point at least touched on by the author regarding the relationship between cost and value. An

* Kansas City, Mo.

endless amount of confusion seems to be caused by many writers failing to grasp the truth that cost is not value, and indeed may have no relation whatever to value. This fact may perhaps be aptly illustrated by an incident from the experience of the writer in acquiring possession of a supposedly valuable puppy, 3 months old. The former owner priced the pup at $25, and convinced the writer that this was the value of the animal, whereupon ensued a transfer of cash and ownership of the pup. On taking possession of the beast, his first official act was to pick up and devour a $20 bill, the cost of the pup automatically nearly doubling without any visible increase in value. In fact, there was at that particular moment much doubt in the mind of the writer whether he really possessed any value whatever, but, as the months passed, the pup developed into a respectable dog, at a cost about normal for this class of development expense, and has since taken prizes more than sufficient to pay his development expenses, including the original purchase price and the initial $20 feast. His value at the present time is considered to be in excess of, and entirely independent of, his past cost. **Mr. Burns.**

Public utilities are very similar in many ways to this dog. Their development expense may begin with a startling expense account, entirely unlooked for, after which they perhaps settle down to a normal development, and steadily appreciate in value. Then, when the time comes for an appraisal of the utility, the owner is confronted with the possibility of having its value judged by an appraiser who may hold the view that past cost represents value, a theory that would clearly disqualify its advocate from holding a position of judge at a dog show.

Passing now to the "conceptual process" outlined by the author: the writer is impressed with the statement that much of the cost of reproduction will depend on the time assumed for the practical operations. This is particularly true of many of the owners' expenses for organization, administration, interest during construction, and similar items, as it is the writer's experience that the time usually assumed for construction work is much too short, due to the time that it has actually required in much of the larger work with which the writer has been identified. This is especially true as to the time required for preliminary investigations and financing of public enterprises.

Another point occurs to the writer as worthy of special emphasis is suggested by the author's "summary of reproduction". He states:

"Ninth.—If a plant is carefully and honestly built, if it serves the need in the community in the best possible manner, and if there is no better or less expensive service, then the cost of reproduction must of necessity be the least possible value that can be put on the utility, but the real value may often be in that case more than the reproduction cost."

Mr. Burns.

This statement offers an opportunity for a serious consideration of a far-reaching point, affecting not only the reproduction cost of a public utility, but in many cases its past cost as well. It is sometimes argued by members of the legal fraternity that certain portions of the property of an over-built plant are not properly included in a valuation, on the theory that the law limits them to a consideration of property used and useful. Hence, they argue, some of the real estate perhaps held in anticipation of future requirements, or an extra liberal provision in plant capacity in a rapidly growing community, not being used or necessary in the plant at the present time, is not properly included in the valuation. Now, if it be granted that such an argument is sound and in accordance with the proper interpretation of the law, it occurs to the writer that it must follow that the allowance for interest during construction must be correspondingly increased, as shown by the following example illustrating the case:

Assume that a property is found to be 25% over-built, and wisely so, in anticipation of future requirements. For example, let it be assumed that a property is found to have a reproduction cost of \$1 250 000, of which amount \$1 000 000 is all that is required for present use, but the additional \$250 000 will be called into use and required to serve the public, say, 5 years hence. In nearly all communities where such a condition exists, it is a fact that the public utility property has passed through a similar condition throughout its entire history; therefore, when the whole property comes into use 5 years hence, its reproduction value should include an extra allowance of 5 years' interest during construction on this \$250 000 investment, because, from the very nature of the investment in the growing community, this cost had to be incurred 5 years sooner that it could be put into use. This \$250 000 investment, when finally transferred to the used and useful class, therefore, should include 25 or 30% additional for interest during construction. Therefore, in estimating the cost of reproduction of such a property, where the conditions of growth are such as to warrant the belief that the property should keep 25% in advance of the immediate demands of the public, then this extra allowance for interest during construction must be included in any estimate of the reproduction cost. Similarly, this being true of the \$250 000 investment now being questioned by the aforesaid attorneys, it is likewise true of the whole \$1 000 000 property now admittedly used and useful, this entire \$1 000 000 property having been similarly built piecemeal, and maintained 5 years ahead of time, all in the interest of the public welfare. This is a point which, as far as the writer is aware, has never been presented in any of the literature on appraisal and valuation work that has come to his attention. However, it occurs to him that, in a situation where the requirements of the public seem to demand that the public utility property be kept

pretty well in advance of the times, this question of interest during construction for a period of time equal to the average over-built condition of the plant should be allowed in all estimates of cost of reproduction, and is, therefore, an element of value to that extent. Mr. Burns.

JOHN W. ALVORD,* M. AM. SOC. C. E. (by letter).—The writer is greatly indebted to the members of the Society who have discussed his paper. What little value there was in his own effort has been greatly enhanced, not only by the discussions which have agreed with his views, and the kindly approval generously accorded, but, as well, by members having differing view-points, who have taken the time and pains to register their honest dissent. All such contribution must cause the final truth to be more sharply thrown into relief. Mr. Alvord.

This paper was written primarily to confine attention to the subject of valuation. Rate-making was avoided as far as possible, so as to limit attention to one subject. So much of the discussion, however, has inevitably drifted into the rate-making results of valuation that it may not be amiss to outline briefly some principles in utilizing valuation for rate-making purposes.

It is desirable to confine such statements to certain fundamentals as to which there is little or no room for controversy, and little or no controversy, in fact. This will tend to clear the ground for a consideration of the details presented by the discussion.

1. Rate-Making.—(*a*) This is a legislative function performed by legislatures, or by commissions or municipalities to which the legislatures have delegated administrative functions in connection with the exercise of this power. This is not a judicial function. Courts have not the power to make rates, and do not make rates.

(*b*) The power to regulate rates is subject to the limitation that the rates shall be reasonable, that is, that they shall not exceed the fair value of the service to the consumer, and shall not be less than that which will afford a fair return on the value of the property.

(*c*) Reasonable rates are such as are just and fair, between the limits stated, after due consideration of all the pertinent circumstances.

The maximum of what the service is worth to the consumer and the minimum of what will afford a fair return to the utility constitute the limits within which the rate-making power may justly exercise "the flexible limit of judgment which belongs to the power to fix rates". (206 U. S., 26.) Different conclusions, both reasonable, may be reached by equally competent tribunals from the same facts. There is no mathematical rule.

"With that sort of evidence before them, rate experts of acknowledged ability and fairness, and each acting independently of the other, may not have reached identically the same conclusion. We do not

* Chicago, Ill.

Mr. Alvord. know whether the results would have been approximately the same. For there is no possibility of solving the question as though it were a mathematical problem to which there could only be one correct answer." (222 U. S., 550.)

(*d*) Economic laws require value to be taken into account in performing the legislative function of rate-making. The money for investment in public utilities must be secured in a competitive market. It can only be induced to go into these utilities by the promise of a sufficiently attractive return. Practically every public utility constantly requires new money for extensions and additions. The question is not what return should a commission permit on money which is invested and cannot be withdrawn. It is rather, what return on the money already invested is required to attract the necessary new capital, if the public service is to remain unimpaired. Obviously, fundamental economic laws require, in the long run, that the return from these investments be equal to that which might be derived from other available investments, after adjusting differences of hazard and other variations in the surrounding circumstances.

2. *Suits Assailing Rates.*—The Constitution of the United States, as interpreted by the Courts, prohibits, as involving the taking of property without due process of law, rate schedules which will not afford a fair return.

(*a*) The limit of the jurisdiction of the Courts is to determine whether any specific rate schedule will afford such a return on the fair value of the property at the time in question, and if it will not, to set that schedule aside. The Courts have not the power to prescribe schedules of reasonable rates, or to establish schedules of reasonable rates to take the place of those which are set aside as unreasonable.

(*b*) In a proceeding before the Court, the fair value of the property used for the convenience of the public, or, to put it another way, the reasonable value of the property at the time it is being used for the public, is one of the ultimate facts which must be determined. As to this, it is said by the United States Supreme Court (230 U. S., 434):

"The ascertainment of that value is not controlled by artificial rules. It is not a matter of formulas, but there must be a reasonable judgment, having its basis in a proper consideration of all relevant facts."

A proper appreciation of the radical distinction between the jurisdiction of the rate-making body on the one hand, and of the Courts on the other, will strongly tend to clarify the situation. Perhaps it may be said, without impropriety, that the jurisdiction of the regulating bodies ends where that of the Courts begins. The jurisdiction of the Courts is above the line of confiscation. The reasonable rate lies between the rate that is on the line of confiscation and the rate

that is so high that the charge exceeds the value of the service. The jurisdiction of the Courts may be invoked when the regulating body has exceeded its jurisdiction, and has attempted to establish a rate that is confiscatory. Mr. Alvord.

Within its jurisdiction, the regulating body has the right to take into account every factor which may legitimately affect the exercise of its discretion. The regulating body may, and should, take into account, not only the value at the time, but also the investment, what it would cost to reproduce the property new, the probable investments that will be required in the future, whether the service rendered by the plant is of 100% efficiency, the revenues which the plant has been earning and which it would probably earn under the proposed schedule of rates, and every other fact material to the determination of what, as between the utility on the one hand and the consumer on the other, would be reasonable and just.

Much of the confusion with reference to this matter has apparently grown out of the assumption that the regulating body has the power to take into account, in connection with the investment, only a single factor, which must be called value, whatever it is. Partly on this account, some who have believed that original cost should be the basis of rates, have advocated this as a measure of value; others, for the same reason, have advocated 100% of the reproduction new; others have advocated reproduction new less depreciation; and others have presented still different theories. As a matter of fact, every one of these factors may be material in connection with the establishment of a rate schedule, and may properly be considered under its own proper name, without an attempt to make it available through a forced definition of the word "value".

On the other hand, when we go out of the domain of rate-regulation into the jurisdiction of the Courts, the rule is perfectly well established that the material fact is the value of the property, and the Courts have properly said that this cannot be determined by any formula. There is no formula which will show the value of a horse, or of a piece of real estate, or of any other property. In every case, there must be "reasonable judgment, having its basis in a proper consideration of all relevant facts". In announcing this rule, the Court has simply declared what is common experience. Value is, and should be, used in its ordinary, every-day sense. It is difficult to define it, but it is no more difficult than to define what is the value of a horse, or of a piece of real estate, although the determination of what is the value of the utility may present a much more difficult and complex question.

A careful consideration of these fundamental principles will clear away much of the confusion of mind observable in some of the discussion on this and the writer's prior paper, "The Depreciation of Public

Mr. Alvord.

Utility Properties as Affecting Their Valuation and Fair Return";* in fact, this present paper should have preceded the former one, in order that both might be intelligible.

Turning now to some of the discussions of the present paper in detail, the writer has naturally been particularly interested in those which have taken issue with the paper, and though he regrets the necessity of the "last word" imposed on him by the closure, nevertheless, he would hope that some remark, particularly on some of the important dissenting discussions, will not be without further helpfulness in the matter.

Mr. Haupt's review of the Monongahela Navigation Company's case is of great interest, for it is to be remembered that this is one of the leading cases in the Supreme Court on franchise value. (Monongahela Navigation Co. *vs.* U. S., 148 U. S., 312.)

Mr. Jewel, Mr. Dana, and Mr. Lavis, though cordially assenting to the writer's views, have each and all made valuable criticisms and suggestions which are appreciated.

The discussion by Mr. Mayer is interesting and valuable. He has come to the subject from the standpoint of the theoretical and logical economist, and the writer has read what he has to say with interest and profit. A second reading, however, has raised some question as to the soundness of some of his propositions, and caused some regret that he is not as helpfully constructive in his economic ideas as might be desired. For instance, he criticises all the definitions of value offered, and it must be admitted that most of his criticisms have force, but he does not offer any definition of value to take the place of those he apparently destroys, and one might be puzzled to know, after a careful reading, what his brief definition of value would be. The writer concludes, after reviewing Mr. Mayer's closing discussion† on his paper, "The Just Value of Monopolies, and the Regulation of the Prices of Their Products", that it may be something like this:

"Value is a measure of the supply required for a human need."

Perhaps this is a composite of what is in the minds of Mr. Mayer and the writer, or perhaps it is wholly a pre-occupation of the writer's. Be this as it may, it is offered for what it is worth.

The writer has been unable to grasp Mr. Mayer's criticism on his own adopted definition of value (page 178), that it is contradictory, although he has read over and thought about the criticism not a little.

Mr. Mayer thinks that an important factor has been omitted, in considering values, by neglecting to mention the importance of the character of the future regulation. It would undoubtedly add much, not only to our ability to value correctly, but to our ability to form

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 788.

† *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 492.

proper judgments in all other aspects of life, were we certainly apprised of that interesting class of facts which lie wholly in the future; but what the character of future regulation will be is a factor which no one can ever certainly know, and, therefore, it must be a speculation in any case. It is this fact, more than perhaps any other, that causes value to be a matter of opinion. All value is predicated fundamentally on what the future will bring forth, as Mr. Mayer suggests, and perhaps it would have been well if the paper had dwelt on that fact to a greater extent, but the paper is not a thesis on theoretical economics, it is an attempt to point out the practical governing conditions under which we must proceed to accomplish something specific in the light of our restricted knowledge, and, for that reason, we must build on what we have. Mr. Alvord.

Other suggestions of Mr. Mayer might be commented on as follows. He very properly says:

(*a*) That the foundation of valuation rests on equity and justice. (Page 177.) This is quite true, but the formulated equity and justice under which we must actually work are the Constitution, the law, and the interpretation by the Courts, which are serious and able attempts to make equity and justice practically workable, and we do not want to take time now to consider what would happen if we did not have these valuable precedents and rules of practice in our work.

(*b*) Future legislation does not concern us, because, as Mr. Mayer says, we cannot know what it is to be. (Page 182.) We must assume from the past, therefore, that it will be reasonable and fair, so as to attract capital into the utility field.

(*c*) It is not especially valuable to us to know that every utility, even of the same kind, should have a theoretical fair return, different from any other utility, and predicated on what like competitive business would produce. This may be interesting theoretically, but all we can hope to accomplish practically is some approach to justice between the buyer and seller of specific service, which can be approximated by comparatively simple and reasonable methods in each instance. (Page 187.)

(*d*) In holding that it would be absurd to attempt to find the depreciated value of an obsolete plant by the reproduction process, Mr. Mayer goes to an extreme case to prove his point. (Page 184.) It is unusual to find a wholly obsolete plant in service. Normally, plants are only partly obsolete, and usually only slightly so, and it is certainly of great advantage to go through the reproduction process in such cases, as it affords much valuable insight into structural value to examine a plant in detail.

(*e*) Mr. Mayer says: The true worth of any utility is the present worth of its future net earnings. (Page 185.) Presumably this is

Mr. Alvord.

sound, if the physical plant is amortized at the end of the earning period, but of what interest is it to us practically; we can never hope to know by any possibility the present worth of the future net earnings. What looms large to us as a sheet-anchor, in our attempt to lay hold of something practically helpful, is that most utilities have the largest portion of their investment in the physical plant, and it is approximately possible to tell what this plant is worth, judging by the need for its output, and it is also possible to tell what it will cost to finance it on a paying basis, also, in some cases, what it has cost. With these facts and other surrounding circumstances, it is further possible to answer approximately the question, what is its present fair value, always supposing, of course, that it is not to be destroyed by unjust legislation, wrecked by careless management, or engulfed by an earthquake.

Mr. Mayer has said many valuable things, and has pointed out particularly well, in the writer's opinion:

(1) The fallacy of public regulation of rates, without incentive to good management;

(2) The importance of comparing "fair return" with that obtained in kindred enterprises; and

(3) The fact that the present tendencies of regulation are inevitably leading to guaranteeing the investment and operating the property by the commissions.

Mr. Burgess has presented a very effective plea for more deference to evidence of past cost, and the writer would cheerfully acknowledge that, under most normal conditions, much of what he says is reasonable, and under certain exceptional conditions, even more could be said in behalf of past cost.

It is very difficult, in such a discussion as this, to select the proper perspective for the consideration of two cost methods, each of which will vary in preponderance in different cases. Past-cost evidence has undoubtedly been over-emphasized, by some of our commissions, where it should not have been, and, in an attempt to offset this tendency, the paper may not have adequately conveyed the impression which the writer really holds, that past cost always has some remote usefulness in valuation work, and has an undoubted value where it is not too remote from present conditions. The nearer past cost is to the present time, the more and more valuable it becomes, until, finally, in considering a plant which has just been built, past cost, of course, must have a large preponderating influence in the summing up. The writer had occasion recently to value two water-works plants; one had been built within 8 years, the other was nearly 100 years old. The past cost was known in both cases, but the older plant, originally built with

bored logs, had no trace of its original construction left. Obviously, past cost, under these conditions, did not throw much light on present value; but, in the other case, past cost was so recent and so complete, that reproduction cost new would have been little different in result. Mr. Alvord.

Mr. Burgess has made it clear that, in very old plants, past cost should not have too much weight, and it is believed that there is no essential difference between his views as stated, and those embodied in the paper, except that the perspective in the paper on this point is perhaps not as fully and well brought out as it might have been.

Mr. Newell, in his brief discussion, makes the following astonishing statement: "Under regulation, the duty of the public service commission is to determine, not what the value of the property is, but what it ought to be."

The writer cannot help reflecting what a beautiful world this would be if we, as self-appointed commissions, could determine the value of our individual property, and fix it, not on the basis of what it is, but what it ought to be.

Mr. Harte, as well as Mr. Mayer and Mr. Dana, has criticised very intelligently the definitions which the writer quoted from legal authorities on valuation. It is realized that these definitions are not as satisfactory as could be wished. Possibly "value" is not susceptible of a clear, brief definition, but it would seem to be desirable that one should be determined, if possible. Thus far, the writer has been less satisfied with those definitions which are found in books on economics than with those which he has submitted from legal opinions.

In the writer's mind, the difficulty which disturbs such men as Mr. Humphreys, Mr. Grunsky, and Mr. Knox (pages 199 and 213), who, in brief, believe depreciation should not be deducted from cost new, and that rates should be adjusted to 100% value, is this: That they have approached the problem of valuation from the view-point that the value of the property is of controlling importance in fixing rates, and that the commissions must and should protect the original investment, whereas the view-point of the law is that it is the value of the property now that is considered only as a lower limit beyond which there must not be confiscation.

They neglect to observe that the value of the property now is useful chiefly in defining this lower limit of fair return, as distinguished from confiscation, and that the field, or fair rates, lies between the line of confiscatory return, on the one hand, and the worth of the service to the consumer, as an upper limit, on the other.

If the State were to undertake, by suitable rates only, to protect the original investment intact, without allowing it to grow by so-called "unearned increments", or to lessen by such losses as may occur, then it is obvious, as these gentlemen contend, that depreciation should not be deducted from the original cost for rate-making purposes.

Mr. Alvord. The writer, however, has attempted to show in the present paper, that no such condition of protection by the State of an original investment is contemplated by the law under which we are working. What is expected under the rule now in use is that the original investor shall accept all the inevitable ups and downs to which his original investment may be subjected, and that the rates, where the rate-making powers are exercised, shall lie within the field limited, on the one hand, by the worth of the service to the consumer, and, on the other, by the necessity of avoiding confiscatory return on the fair value of the property at the time such powers are invoked. At such time, this fair value must, therefore, under this theory, include all the appreciations as well as exclude all the depreciations to which the property has been subjected since the original investment was made.

Mr. Humphreys argues (page 203) because renewals are numerous and frequent in some kinds of property, and, owing to short life and small individual amount, such depreciations are, for convenience, converted into operating expenses, that the property as a whole ought to be considered as having original value without depreciation deduction. This may be a practical way to handle some forms of depreciation in certain extreme cases, but it does not alter the general principle that losses of value which have occurred since the original investment was made should be considered, together with increases in those values, in determining the value of the property at any given date.

For example, if a prosperous and growing main-line railroad has a short but expensive branch to a mine which becomes worked out and ceases to operate, and this branch, in consequence, has become of no further use, present or prospective, we speak of it as having depreciated in value, and we must take that fact into consideration, as well as the increased value of the main line, in arriving at the value of the railroad property as a whole as of the date of valuation.

If, in the foregoing case, by agreement with the State, we were considering only the protection of the original investment, we should have to continue to include the branch at its original investment cost, and deduct from the main line all the increased value it had acquired by its prosperous growth.

The unwisdom of considering the State as a guarantor of investment, through its rate-regulating powers, is too obvious to be discussed at length. The State cannot engage in a policy of protecting investment, even by the roundabout method of establishing fair return on original cost, as a settled policy. To do this would be to take all incentive for economy or prudence from the private investor, and to assume all the uncertainties and vicissitudes of an unknown future in the business world. Hence the rule has been adopted by the Courts, in interpreting the Constitution and the law, that it is the fair value of

the property as of the date when such rates must be determined, and it requires little reflection to see that any other course would not be sound or wise public policy conducive to justice and the public welfare. Mr. Alvord.

Mr. Knox continues, in his discussion, his dissent to the writer's former paper* on depreciation, although, in the light of the closure to that paper, and the present paper, the modification to his thought is apparent.

Among other matters, in the course of his argument, Mr. Knox strives to create what seems to the writer to be a new definition of depreciation, namely, that it is limited to physical wear and tear only. This idea seems to prepossess a number of engineers writing on this subject, but is certainly without warrant. Webster gives no such restricted definition. Common use does not warrant such a limitation. When we speak of a house depreciating, we do not consider that it is lessened in worth only by physical wear and tear. Age, changing styles, lessened demand, and other causes have usually much more to do with its loss of value than wear and tear.

Practical work in appraisals very quickly convinces one that lessened value arises from many causes other than wear or physical deterioration, and these other causes have usually much greater influence in depreciating property. Take, for instance, the illustration previously used, of the railway having a branch to a worked-out mine. We could assume that the branch line, not anticipating abandonment, had recently been renewed as to its rails, ties, and bridges, so that it was physically as good as new. This would not have prevented its large loss of value as a branch, due to lack of need for its facilities. The depreciation might have been a little greater if the ties and rails were old, because the scrap value would have been less, but the main factor of loss is to be found in changed conditions of general need for the service, and all the arguments that Mr. Knox has suggested do not alter the condition that if we must value such a property, and value it as of the present time, we must deduct, as a depreciation, the loss of value of the branch line in considering the fair present value of the railroad as a whole.

Mr. Knox (page 209) can find no instance in which the Courts have decided that depreciation reserves are to be computed on the sinking-fund basis, or that "physical" depreciation must be deducted from value new in rate-making appraisals.

If Mr. Knox expects to find such explicit directions as to engineering and economic details of valuation, such as the use of sinking funds, from the Courts, he will be disappointed. The Courts do not usually presume to go so far outside of their proper domain. Engineers sometimes unconsciously presume to instruct the Courts as to

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 788.

Mr. Alvord. law, but few Courts presume to instruct engineers as to detailed engineering methods. The Courts interpret the law and pass on the evidence, but it is for the engineer and the economist to formulate rational methods of presenting the evidence.

As to how loss of value in structures and property is to be determined, is, of course, an interesting question, on which opinions can honestly differ. The assumption that depreciation as a loss in value is often well measured by the accumulation of an amortization fund, is a method of basing such estimating which might be properly debated and discussed. Obviously, it is not true in every instance, or in every kind of a utility, or in any single structure at every point in its career; but, as an attempt to average a great variety of conditions, it has hitherto been accepted by a large majority of appraisers as fairly well answering general purposes as a beginning from which to reason. To the writer's knowledge, no one hitherto has accepted it as a hard and fast rule or principle. Possibly, this tendency to accept it as a mathematical solution has also troubled some of those who study depreciation discussions.

The question of deducting depreciation from cost new, however, is fundamental to a proper value, and, as a matter of fact, for the past 15 years, engineers and attorneys have been arguing for its deduction, and Courts have been approving the practice of subtracting the depreciation from reproduction cost new in every case brought before them, without a single exception which the writer can recall. Engineering appraisal boards, so far as the writer is aware, have pursued the same practices. In water-works appraisal, the question was discussed and settled more than 15 years ago, and has never been seriously controverted since, until Mr. Grunsky's paper, "The Appraisal of Public Service Properties as a Basis for the Regulation of Rates",* appeared, in 1912, before this Society. That paper ignored the settled practice, so carefully thought out and already well argued and passed on, and it seemed to the writer that the original reasoning by which the question had been settled should be stated for record. This he did in a former paper† before the Society. Mr. Grunsky argues in his discussion (page 213) that as net profits fix the value of a property, the value should not be used in fixing rates, for this is reasoning in a circle. From this, he apparently argues that the whole present method of fixing value is wrong, and Court decisions based on it are erroneous and should be changed.

Mr. Grunsky must realize that, in taking such a position, he is passing adversely on a proposition which has been settled by a large

* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 770.

† "The Depreciation of Public Utility Properties as Affecting Their Valuation, and Fair Return", *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 788.

number of trained minds—official, judicial, and expert—who have considered this problem carefully for many years. Mr. Alvord.

The writer has endeavored to point out in his paper (page 134) that we do not reason in a circle by fixing value first, because we have, in all ordinary types of utilities, a physical plant as a basis, and this, together with a fair return (which is assumed to be sufficient to cause capital to flow freely into the business), will fix the fair value of the property, and fix it so effectively that there can be only one value for a given physical plant and a given minimum fair return. This fundamental proposition was the subject of much discussion among appraisers some years ago, but was found to be capable of simple solution, and has in no wise troubled Courts or appraisers since. Hence, we need not overturn anything—not even so well established a practice as that of the Supreme Court—in the effort to establish justice.

Mr. Kiersted frankly dissents from some of the principles which the paper has endeavored to point out as fundamental. Briefly summarized as to his important points, he would seem to hold:

(*a*) That equity and justice are the foundations of valuation work.

(*b*) That the reproduction theory really embraces three distinct methods of valuation:

(1).—Present-day conditions and prices;

(2).—Original conditions and present-day prices; and

(3).—Original conditions and original prices.

(*c*) That there may be no practical distinction between cost and value in monopolistic utilities.

(*d*) That the determination of value for purchase and sale may be different from that for rate-making.

For the most part, the paper itself is an attempt to answer these positions. Mr. Kiersted's view-point being obviously that it is an investment which desirably must be protected and assured, apparently he does not see the usefulness of the law's attitude that the fair value of the property as of to-day should be the governing factor.

Mr. Burns' discussion contains a very good illustration of fluctuating value, which seems to answer Mr. Kiersted's contention that cost and value may be practically identical. Something, however, may be said briefly as to the other points raised:

(*a*) Equity and justice are most assuredly the foundation of appraisal work, but everybody's idea of equity and justice is, unfortunately, not the same, consequently, the American people have formulated their fundamental ideas of equity and justice into a Constitution and laws, which, as interpreted by the Courts, are guides in cases of

Mr. Alvord. differing opinion. It is obvious that we must work under these formulas until they are changed by general consent.

(*b*) In outlining three methods by which reproduction processes may be carried out, Mr. Kiersted uses the term "reproduction" in an apparently different sense than it is ordinarily used. He says (page 215): "The reproduction theory really embraces three distinct methods of valuation," which he goes on to describe.

The writer's definition of reproduction (page 120) is: "An estimate of the cost of re-creating a property at the present time under conditions that are humanly possible and practicable."

Mr. Kiersted's conception, as nearly as can be defined from an examination, especially of the second and third methods which he outlines for its use, would seem to be: The re-creation of the whole or a part of the whole of a given property, without reference to physical possibilities or human limitations.

In the first case which he outlines—that of re-creating a property under present-day conditions and prices—he fairly well describes reproduction as appraisers generally understand it, but, in the second method cited, he outlines a form of re-creation such as is not, humanly speaking, possible, namely, the re-creation under original conditions, but at present-day prices. Mr. Kiersted has had a long and successful experience in building and remodeling water-works properties, but he certainly would not claim to be capable of building plants at prices to-day under conditions which existed 20 or 30 years ago.

An engineer who could construct the New York Central Terminal, in New York City, at present-day prices, but under conditions of population and land values which existed when the old New York and Harlem Railroad was first projected, would promptly have his hands full of business from thousands of clients.

Such a so-called reproduction proposition does not aid in finding the value to-day of the New York Central Terminal. It merely confuses the question. What we want to know is either what that terminal has actually cost in investment, with its several re-buildings, all told, in the long years it has been developing, or what it would cost to reproduce it to-day under present conditions. Nothing of a humanly impossible character halfway between these two rational inquiries throws any real light on what it is worth, or helps us in the least to base our ideas of value on workable facts and sound fundamental conditions.

Nor is Mr. Kiersted's third method of so-called reproduction entirely free from the same kind of objection. On the face of it, it is the retracing of the past investment, or, in other words, the attempt to substitute for actual book cost an estimate of book cost which will be complete and presumably as good; but, as Mr. Kiersted defines it, it is

not even the reproduction of the past cost, because, as he says, it would take account only of the property existing at the present time, useful for utility purposes, and thus exclude all obsolete structures or other investment originally necessary, but now non-existent. Mr. Alvord.

The same criticism, therefore, would hold here, that lies against the second so-called reproduction method. It is not original conditions; it is not present conditions; it is (to use Mr. Kiersted's own words) an unreliable phantom that lies between the two and can never be an actuality. It gives us nothing which is humanly possible of accomplishment; nothing for a foundation of reality on which intelligently to build an opinion as to value, and, therefore, it is misleading in so far as it purports to be evidence.

Unfortunately, it is true, as Mr. Kiersted says, that, in the past, many valuations have been a conglomerate of the three methods, but this is a regrettable stage of progress from which we must obviously emerge as promptly as possible into a domain of more rational thought and procedure. Appraisal work in the future must advance to the point where the engineer can give a sound reason for every position that he holds and for every step that he takes, or it will continue to be as unscientific in the future as it has unfortunately been in many cases in the past.

One further point in Mr. Kiersted's discussion remains to be noted, that is, his obvious belief that value for rate-making may be different from business value. This apparently arises from his strong conviction that in appraising a property he is protecting an investment rather than finding its value as of to-day.

If the State, through its utility commission, were proposing to an investor the protection of his original investment by means of fair rates on its actual original cost, then it might easily happen, after a lapse of years, that the artificial value legislated into the property by the rate-making power might create a value for the property greater than its real value without legislative stimulus. This would not mean that the property had two values at the same time, but that it might have widely different values at two different times, dependent on the legislation of the State. The State, however, does not make any such proposition, and it has been suggested, and is very easily seen, that it would not be a safe or wise policy. Imagine a law being passed regulating rents for business blocks in a city to a fair return on the original investment, and the State and the investors contracting with each other to the end that, in view of regulated rents, the State would accept the original investments as a safe continuing basis for fair return!

Suppose that in one-half of the city there should be a great shrinkage of values due to the gradual removal of the population to the second half, while, in the second half, there should be a marked

Mr. Alvord. increase in prosperity following the changing conditions; what would happen? The property in that part of the city of lessening value—the first half—would be depopulated faster than ever, and the State would be unable to fulfill its part of the agreement, but in the second half—or prosperous part of town—great injustice would have been done to the owner in depriving him of his enhanced values.

In a similar manner, a State cannot assume the policy of fixing rates on a large number of public utilities within its borders on the theory that it will protect their original investment through adequate returns. If it does, it will do itself an injustice in the case of those properties having diminishing values, as well as do injustice to owners of those utilities which may come to have increasing values.

As has been said before, it is for these reasons, as well as others of equal importance, that, in the regulation of utilities, the burden or reward, or loss or gain in value of a property through a term of years, must, as a matter of sound policy, rest on the private owner. Hence the rule that the value of the property as of to-day is to be the governing factor.

If the value of the property as of to-day is the governing factor in fixing rates, there can be only one value for rate-making, or for sale to the city, or business purposes, and the test of this truth lies in the fact that all such properties live and grow by reason of the new capital that has already flowed and should continue to flow into them; and, in the long run, the fair return must be fixed with the end in view that capital will be attracted to the utility with the expectation that it will earn a return adequate to the obligations and hazards involved, and, at the same time, have opportunity of reasonable increase in value. When this is the case (and it must normally be the case, or utilities would not exist), the value for rate-making is also the value for either sale or purchase; for, were it not the case, such property would in time sicken and die if just rates were not replaced.

To confirm this truth further, let it be imagined, if possible, that a utility property had two values at the same time, one for rate-making purposes and one for sale purposes. Who would pay the value for sale purposes if it was materially higher than that on which the rates would yield a fair return; or who would willingly sell a plant if the sale value was arbitrarily fixed lower than the rates indicated was profitable? Evidently, value can be legislated temporarily into or out of a public utility property to some extent, but two values for the same property under the same conditions is apparently unthinkable.

In conclusion, the writer hopes that those who have so kindly discussed his paper will take in good part his earnestness in defending the point of view that we must not neglect to study the legal aspect

of valuation if we would do constructive work in valuation matters. It is not only the writer's view, but is that of all the ablest engineers, economists, and lawyers with whom he has been associated, and he will willingly confess that for many years valuation as an art had no co-ordination for him as a purely engineering problem. It was not until he began to get an inkling of the fundamental ideas of justice, as embodied in the law and its theory, that he began to perceive that valuation as an art had underlying principles which gave it unity and intelligent purpose. Mr. Alvord.

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TRANSACTIONS

Paper No. 1337

PENSTOCK AND SURGE-TANK PROBLEMS*

BY MINTON M. WARREN, JUN. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. IRVING P. CHURCH, R. D. JOHNSON, JOHN C. TRAUTWINE, JR., GARDNER S. WILLIAMS, J. T. NOBLE ANDERSON, H. C. VENSANO, AND MINTON M. WARREN.

SYNOPSIS.

Object.—The object of this paper is to give, in their best form, formulas for solving certain penstock and surge-tank problems, and to derive them from the fundamental principles on which they rest. In the case of ordinary water-hammer, for instance, for which every pipe should be investigated, there is no literature in English except certain incomplete translations from the Italian and Russian. Most of the formulas given in standard books are either limited in their application, incorrect, or contain typographical errors, making them useless. Their results differ by 100 per cent. It is with the object of clearing up such errors that this paper is presented.

Digest of Paper.—The paper may be classified under three headings:

Water-Hammer;

Surge in Surge Tanks; and

Economical Penstock Size.

Under the heading, "Water-Hammer", the ordinary formula for maximum water-hammer is first derived. A simple formula for ordinary cases, when the gate is not closed instantaneously, is then given.

* Presented at the meeting of December 2d, 1914.

The formulas of other writers for this case are also derived, and their limitations and inaccuracies are shown by numerical examples.

Under the heading, "Surge in Surge Tanks", a new formula is given, with a mathematical derivation which it is hoped will be readily understood and followed.

A simple formula for Economical Pipe Size is given for use in water-power developments having long pipes or penstocks.

Conclusions.—Although water-hammer and surge-tank design are common problems to the hydraulic engineer, English literature on these subjects is scattered and incomplete. From constant copying, the formulas have become wrongly applied, and for special cases they are sometimes given as general. Starting with fundamental principles, these problems can be solved by any engineer, and are less complicated than may appear at first sight.

Introduction.

The purpose of this paper is, first, to give in their simplest form for ready reference the best formulas for solving certain surge-tank and penstock problems; and second, for the benefit of engineers who do not believe a formula merely because they see it in print, completely to derive each formula from the fundamental principles on which it rests. All the corresponding formulas which have been found in current literature are then given, and finally numerical examples are worked out, with a comparison of numerical results, using all the different methods. A bibliography of the subject is appended.

Many of these formulas have been published before in standard works, but the writer has found in many cases typographical and other errors that rendered them misleading or incorrect. In most cases no derivation is given, and therefore such errors are difficult to locate.

Formula (B) for ordinary water-hammer, Formula (G)* for surge tanks, and Formula (H) for economical pipe diameter have not been presented before, as far as the writer knows. Moreover, the usual general formulas for water-hammer given in standard works all appear to be more or less limited in their application.

*Since the foregoing was written, a series of articles on surge tanks, by Professor Prasil, has appeared in *The Canadian Engineer*, in which the method of attack is very much the same.

The writing of this paper was prompted by the fact that, in investigating the safety of a certain pipe design against water-hammer, the formulas given by one authority showed it to be safe, and another formula indicated that the pipe thickness should be doubled. All the best literature on the subject was found to be in Italian or Russian.

On comparing the formulas from all sources, it appeared that one, Formula (A), on which all writers agreed, applied up to a certain critical time. Beyond this time another formula applied. At the critical time alone, both formulas held.

On testing the different formulas for this critical time, the results differed widely and were inconsistent with Formula (A). Some gave impossible results above the theoretical maximum; with others the results were half as large.

This showed that some of the formulas were fundamentally wrong, and for this reason a rational general formula was sought.

The following is not intended to be an exhaustive treatise on the subject of penstocks and surge tanks. Certain complications, such as governor action, are omitted altogether. The formulas are derived from the maximum values, in which the engineer is most interested, and against which he has to design. Formulas for cases in which only part load is thrown off or on are not given, as the results are less, numerically, and the complication of dealing with two velocities is considerable. In the case of starting or stopping, one of these velocities becomes zero. The mathematics has been simplified as much as possible, and all assumptions have been clearly stated.

The derivations are given, not merely for the value of the resulting formulas, but as a basis for discussion and a starting point from which other special cases can be worked out.

Nomenclature.

The following gives the meaning of every term used in the formulas. All units are given in feet, pounds, and seconds, but any other consistent system will apply equally well.

A = cross-section area of surge tank, in square feet;

a = velocity of vibration along pipe, in feet per second;

b = thickness of pipe walls, in feet;

B = cost of pipe in place, per foot of diameter per foot of length;

C = coefficient in Chezy formula for friction in pipes;

D = inside diameter of pipe, in feet;

E = modulus of elasticity of pipe material in tension, taken as 4 000 000 000 lb. per sq. ft. for steel plate (28 000 000 lb. per sq. in.);

e = 2.718, base of Napierian logarithms;

F = total feet of head lost in pipe between reservoir and standpipe with Q cu. ft. per sec. flowing;

g = acceleration of gravity = 32.2 ft. per sec. per sec.;

h = head due to water-hammer alone (in excess of static head), in feet;

i = income, in dollars per year per foot of head;

K = modulus of elasticity of water in compression, taken as 42 400 000 lb. per sq. ft. (294 000 lb. per sq. in.);

L = length of pipe, in feet;

$m = \dfrac{F}{2\,Q}\sqrt{\dfrac{A\,g\,P}{L}}$;

$m' = \dfrac{m}{\sqrt{1-m^2}}$;

$N = \left(\dfrac{L\,V}{g\,T\,Y}\right)^2$;

p = percentage return on investment, including profit, depreciation, maintenance, and taxes;

P = cross-sectional area of pipe, in square feet;

Q = steady flow through pipe, in cubic feet per second; in stopping, before gate starts to close; in starting, after gate is open and equilibrium is established;

q = square root of mean square of flow through penstock, in cubic feet per second. (See discussion under Economical Penstock Size.)

S = maximum surge, up or down, in feet, measured, in starting, from reservoir level; and, in stopping, from a distance below this equal to the friction head, F;

s = distance of surge-tank level from starting point at any time, in feet;

T = time of closing of gate, in seconds;

t = time after starting to close or open gate, in seconds;

t_1 = time to reach maximum draw-down, in seconds;

τ = time for a complete cycle of oscillations in the surge tank, in seconds;

u = velocity through gate at any time, in feet per second;

V = velocity of water in pipe, in feet per second corresponding to Q;

v = velocity at any time, in feet per second;

v_0 = velocity near gate at any time, in feet per second;

W = density of water, taken as 62.4 lb. per cu. ft.;

x = distance along pipe, in feet, plus, in the direction from gate to reservoir;

Y = normal static head, in feet;

y = head, in feet, at any point at any time;

y_0 = head, in feet, on gate at any time;

$\psi(t) = \dfrac{\text{area of gate}}{\text{area of pipe}}$, at time, t.

SUMMARY OF FORMULAS.

Maximum Water-Hammer.—$\left(T \textit{ less than } \dfrac{2\,L}{a}\right)$.—The maximum water-hammer which is possible in a pipe will occur with instantaneous closing, or for any closing time less than $\dfrac{2\,L}{a}$. For this case

$$h = \frac{aV}{g} = \frac{145\,V}{\sqrt{1 + \dfrac{K\,D}{E\,b}}} \quad \ldots\ldots\ldots\ldots\ldots (A)$$

$$a = \frac{4\,660}{\sqrt{1 + \dfrac{K\,D}{E\,b}}}$$

For steel pipe these become

$$h = \frac{145\,V}{\sqrt{1 + 0.01\,\dfrac{D}{b}}}$$

$$a = \frac{4\,660}{\sqrt{1 + 0.01\,\dfrac{D}{b}}}$$

Ordinary Water-Hammer.—$\left(T \textit{ greater than } \dfrac{2\,L}{a}\right)$.—For ordinary cases

$$h = \frac{L\,V}{g\left(T - \dfrac{L}{a}\right)} \quad \ldots\ldots\ldots\ldots\ldots (B)$$

Allievi gives the following, which is applicable in slow closing, but may be incorrect for quick closing:

$$\frac{h}{Y} = \frac{N}{2} \pm \sqrt{\frac{N^2}{4} + N} \quad \ldots\ldots (C)$$

$$h = \frac{N\,Y}{2} \pm Y\sqrt{\frac{N^2}{4} + N}$$

$$N = \left(\frac{L\,V}{g\,T\,Y}\right)^2$$

***Surge in Surge Tanks.*—The formulas for the maximum surge in a surge tank, either in starting or stopping the plant, are as follows:**

In starting, S is measured from reservoir level; in stopping, it is measured from a distance below this equal to the friction head.

Neglecting friction

$$S = Q\sqrt{\frac{L}{A\,g\,P}} \quad \ldots\ldots (D)$$

A first approximation, allowing for friction, is

$$S = F + Q\sqrt{\frac{L}{A\,g\,P}} = F\left[1 + \frac{1}{2\,m}\right] \quad \ldots\ldots (E)$$

A closer approximation, allowing for friction, is

$$S = F + \frac{Q\sqrt{\frac{L}{A\,g\,P}}}{e^{\frac{\pi}{2}m}} = F\left[1 + \frac{1}{2\,m\,e^{\frac{\pi}{2}m}}\right] \quad \ldots\ldots (F)$$

where $m = \frac{F}{2\,Q}\sqrt{\frac{A\,g\,P}{L}}$.

The most nearly accurate formula is

$$S = F\left[1 + \frac{1}{2\,me^{m'(\pi - \cos.^{-1} m)}}\right] \quad \ldots\ldots (G)$$

where $m' = \frac{m}{\sqrt{1 - m^2}}$.

***Economical Penstock Size.*—The formula for economical penstock diameter is as follows:**

$$D = \sqrt[6]{\frac{3\,250\,q^2\,i}{C^2\,p\,B}} \quad \ldots\ldots (H)$$

Maximum Water-Hammer.

Derivation of Formula.—Formula (A) may be derived at once from the fundamental concepts of work and energy.

The maximum pressure in the pipe will occur when the moving water is brought to rest instantaneously. Any section of the pipe in which the water is moving contains a known amount of energy, and, in order to bring this water to rest, all this energy must be dissipated in some way.

With instantaneous closing, the only possible outlet for the energy is:

(1) In compressing the water in the pipe,
(2) In distending the pipe.

Writing the expression for these three forms of energy, and simplifying, Formula (A) is obtained in one step.

We assume that all the energy of any section, l, of the pipe goes into distending the pipe and compressing the water in that particular section. The correctness of this assumption may be shown in three ways.

First.—At the gate the first element of water is stopped instantaneously, and it is impossible for it to act on any other section of the pipe than that section; similarly, we can work up the pipe step by step.

Second.—Equation (9A), given later under the heading "Ordinary Water-Hammer", shows that the pressure is propagated up the pipe as a wave without loss of intensity, and, therefore, all parts of the pipe attain the same maximum pressure.

Third.—By experiment with very fast closing of gates, it has been found that the pressure rise is constant at all parts of a given pipe and for practically any length of pipe.

The energy of the moving water in a section of pipe of any length, l, is

$$\text{weight} \times \frac{\text{velocity}^2}{2g}$$

$$= \frac{W \pi D^2 l}{4} \times \frac{V^2}{2g}$$

$$= \frac{W \pi D^2 l V^2}{8g} \text{ foot-pounds} \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

The maximum pressure tending to compress the water in the pipe is

$$Wh \frac{\pi D^2}{4} \text{ (in pounds)},$$

and, as this starts at zero and increases uniformly, the average pressure is half as great.

The distance (measured along the pipe) which l feet of water will be compressed is

$$\frac{Wh\,l}{K} \text{ feet.}$$

The work done in compressing l feet of the water, therefore, is

$$\frac{Wh\,\pi\,D^2}{4} \times \frac{1}{2} \times \frac{Wh\,l}{K}$$

$$= \frac{W^2h^2\,\pi\,l\,D^2}{8\,K} \text{ foot-pounds. } \dots\dots\dots\dots (2)$$

The maximum tension tending to burst a section of pipe of length, l, is

$$\frac{Wh\,l\,D}{2} \text{ pounds, maximum pressure.}$$

As this starts at zero and increases uniformly, the average tension is half of this.

The distance which the ring of steel forming the pipe will stretch is

$$\frac{Wh\,D\,l\,\pi\,D}{2\,b\,E\,l} = \frac{Wh\,\pi\,D^2}{2\,b\,E} \text{ feet.}$$

The work done in stretching this steel, therefore, is

$$\frac{Wh\,D\,l}{2} \times \frac{1}{2} \times \frac{Wh\,\pi\,D^2}{2\,b\,E}$$

$$= \frac{W^2h^2\,\pi\,l\,D^3}{8\,b\,E} \text{ foot-pounds} \dots\dots\dots\dots (3)$$

Setting the expression (1) for the energy in the pipe equal to the two expressions (2) and (3) for the work done by the water in coming to rest, we get

$$\frac{W\,\pi\,l\,D^2\,V^2}{8\,g} = \frac{W^2h^2\,\pi\,l\,D^2}{8\,K} + \frac{W^2h^2\,\pi\,l\,D^3}{8\,b\,E},$$

and simplifying

$$h^2 = \frac{V^2}{W\,g\left(\frac{1}{K} + \frac{D}{b\,E}\right)} \dots\dots\dots\dots (4)$$

$$h = \sqrt{\frac{K}{W\,g}\,\frac{V^2}{\left(1 + \frac{K\,D}{E\,b}\right)}} \dots\dots\dots\dots (5)$$

The writing of this paper was prompted by the fact that, in investigating the safety of a certain pipe design against water-hammer, the formulas given by one authority showed it to be safe, and another formula indicated that the pipe thickness should be doubled. All the best literature on the subject was found to be in Italian or Russian.

On comparing the formulas from all sources, it appeared that one, Formula (A), on which all writers agreed, applied up to a certain critical time. Beyond this time another formula applied. At the critical time alone, both formulas held.

On testing the different formulas for this critical time, the results differed widely and were inconsistent with Formula (A). Some gave impossible results above the theoretical maximum; with others the results were half as large.

This showed that some of the formulas were fundamentally wrong, and for this reason a rational general formula was sought.

The following is not intended to be an exhaustive treatise on the subject of penstocks and surge tanks. Certain complications, such as governor action, are omitted altogether. The formulas are derived from the maximum values, in which the engineer is most interested, and against which he has to design. Formulas for cases in which only part load is thrown off or on are not given, as the results are less, numerically, and the complication of dealing with two velocities is considerable. In the case of starting or stopping, one of these velocities becomes zero. The mathematics has been simplified as much as possible, and all assumptions have been clearly stated.

The derivations are given, not merely for the value of the resulting formulas, but as a basis for discussion and a starting point from which other special cases can be worked out.

NOMENCLATURE.

The following gives the meaning of every term used in the formulas. All units are given in feet, pounds, and seconds, but any other consistent system will apply equally well.

A = cross-section area of surge tank, in square feet;

a = velocity of vibration along pipe, in feet per second;

b = thickness of pipe walls, in feet;

B = cost of pipe in place, per foot of diameter per foot of length;

C = coefficient in Chezy formula for friction in pipes;

D = inside diameter of pipe, in feet;

E = modulus of elasticity of pipe material in tension, taken as 4 000 000 000 lb. per sq. ft. for steel plate (28 000 000 lb. per sq. in.);

e = 2.718, base of Napierian logarithms;

F = total feet of head lost in pipe between reservoir and standpipe with Q cu. ft. per sec. flowing;

g = acceleration of gravity = 32.2 ft. per sec. per sec.;

h = head due to water-hammer alone (in excess of static head), in feet;

i = income, in dollars per year per foot of head;

K = modulus of elasticity of water in compression, taken as 42 400 000 lb. per sq. ft. (294 000 lb. per sq. in.);

L = length of pipe, in feet;

$m = \dfrac{F}{2\,Q}\sqrt{\dfrac{A\,g\,P}{L}}$;

$m' = \dfrac{m}{\sqrt{1 - m^2}}$;

$N = \left(\dfrac{L\,V}{g\,T\,Y}\right)^2$;

p = percentage return on investment, including profit, depreciation, maintenance, and taxes;

P = cross-sectional area of pipe, in square feet;

Q = steady flow through pipe, in cubic feet per second; in stopping, before gate starts to close; in starting, after gate is open and equilibrium is established;

q = square root of mean square of flow through penstock, in cubic feet per second. (See discussion under Economical Penstock Size.)

S = maximum surge, up or down, in feet, measured, in starting, from reservoir level; and, in stopping, from a distance below this equal to the friction head, F;

s = distance of surge-tank level from starting point at any time, in feet;

T = time of closing of gate, in seconds;

t = time after starting to close or open gate, in seconds;

t_1 = time to reach maximum draw-down, in seconds;

τ = time for a complete cycle of oscillations in the surge tank, in seconds;

u = velocity through gate at any time, in feet per second;

V = velocity of water in pipe, in feet per second corresponding to Q;

v = velocity at any time, in feet per second;

v_0 = velocity near gate at any time, in feet per second;

W = density of water, taken as 62.4 lb. per cu. ft.;

x = distance along pipe, in feet, plus, in the direction from gate to reservoir;

Y = normal static head, in feet;

y = head, in feet, at any point at any time;

y_0 = head, in feet, on gate at any time;

$\psi(t) = \dfrac{\text{area of gate}}{\text{area of pipe}}$, at time, t.

Summary of Formulas.

Maximum Water-Hammer.—$\left(T \textit{ less than } \dfrac{2\,L}{a}\right)$.—The maximum water-hammer which is possible in a pipe will occur with instantaneous closing, or for any closing time less than $\dfrac{2\,L}{a}$. For this case

$$h = \frac{a\,V}{g} = \frac{145\,V}{\sqrt{1 + \dfrac{K\,D}{E\,b}}} \qquad \text{(A)}$$

$$a = \frac{4\,660}{\sqrt{1 + \dfrac{K\,D}{E\,b}}}$$

For steel pipe these become

$$h = \frac{145\,V}{\sqrt{1 + 0.01\,\dfrac{D}{b}}}$$

$$a = \frac{4\,660}{\sqrt{1 + 0.01\,\dfrac{D}{b}}}$$

Ordinary Water-Hammer.—$\left(T \textit{ greater than } \dfrac{2\,L}{a}\right)$.—For ordinary cases

$$h = \frac{L\,V}{g\left(T - \dfrac{L}{a}\right)} \qquad \text{(B)}$$

Allievi gives the following, which is applicable in slow closing, but may be incorrect for quick closing:

$$\frac{h}{Y} = \frac{N}{2} \pm \sqrt{\frac{N^2}{4} + N} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (C)$$

$$h = \frac{N\,Y}{2} \pm Y\sqrt{\frac{N^2}{4} + N}$$

$$N = \left(\frac{L\,V}{g\,T\,Y}\right)^2$$

***Surge in Surge Tanks.*—The formulas for the maximum surge in a surge tank, either in starting or stopping the plant, are as follows:**

In starting, S is measured from reservoir level; in stopping, it is measured from a distance below this equal to the friction head.

Neglecting friction

$$S = Q\sqrt{\frac{L}{A\,g\,P}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (D)$$

A first approximation, allowing for friction, is

$$S = F + Q\sqrt{\frac{L}{A\,g\,P}} = F\left[1 + \frac{1}{2\,m}\right] \ldots\ldots\ldots (E)$$

A closer approximation, allowing for friction, is

$$S = F + \frac{Q\sqrt{\dfrac{L}{A\,g\,P}}}{e^{\frac{\pi}{2}m}} = F\left[1 + \frac{1}{2\,m\,e^{\frac{\pi}{2}m}}\right] \ldots\ldots\ldots (F)$$

where $m = \dfrac{F}{2\,Q}\sqrt{\dfrac{A\,g\,P}{L}}.$

The most nearly accurate formula is

$$S = F\left[1 + \frac{1}{2\,m\,e^{m'(\pi - \cos.^{-1} m)}}\right] \ldots\ldots\ldots\ldots\ldots\ldots (G)$$

where $m' = \dfrac{m}{\sqrt{1 - m^2}}.$

***Economical Penstock Size.*—The formula for economical penstock diameter is as follows:**

$$D = \sqrt[6]{\frac{3\,250\,q^2\,i}{C^2\,p\,B}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (H)$$

MAXIMUM WATER-HAMMER.

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With instantaneous closing, the only possible outlet for the energy is:

(1) In compressing the water in the pipe,
(2) In distending the pipe.

Writing the expression for these three forms of energy, and simplifying, Formula (A) is obtained in one step.

We assume that all the energy of any section, l, of the pipe goes into distending the pipe and compressing the water in that particular section. The correctness of this assumption may be shown in three ways.

First.—At the gate the first element of water is stopped instantaneously, and it is impossible for it to act on any other section of the pipe than that section; similarly, we can work up the pipe step by step.

Second.—Equation ($9A$), given later under the heading "Ordinary Water-Hammer", shows that the pressure is propagated up the pipe as a wave without loss of intensity, and, therefore, all parts of the pipe attain the same maximum pressure.

Third.—By experiment with very fast closing of gates, it has been found that the pressure rise is constant at all parts of a given pipe and for practically any length of pipe.

The energy of the moving water in a section of pipe of any length, l, is

$$\text{weight} \times \frac{\text{velocity}^2}{2g}$$

$$= \frac{W\pi D^2 l}{4} \times \frac{V^2}{2g}$$

$$= \frac{W\pi D^2 l V^2}{8g} \text{ foot-pounds} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

The maximum pressure tending to compress the water in the pipe is

$$Wh\frac{\pi D^2}{4} \text{ (in pounds)},$$

and, as this starts at zero and increases uniformly, the average pressure is half as great.

The distance (measured along the pipe) which l feet of water will be compressed is

$$\frac{W h\, l}{K} \text{ feet.}$$

The work done in compressing l feet of the water, therefore, is

$$\frac{W h\, \pi D^2}{4} \times \frac{1}{2} \times \frac{W h\, l}{K}$$

$$= \frac{W^2 h^2\, \pi\, l\, D^2}{8\, K} \text{ foot-pounds.} \qquad (2)$$

The maximum tension tending to burst a section of pipe of length, l, is

$$\frac{W h\, l\, D}{2} \text{ pounds, maximum pressure.}$$

As this starts at zero and increases uniformly, the average tension is half of this.

The distance which the ring of steel forming the pipe will stretch is

$$\frac{W h\, D\, l\, \pi\, D}{2\, b\, E\, l} = \frac{W h\, \pi D^2}{2\, b\, E} \text{ feet.}$$

The work done in stretching this steel, therefore, is

$$\frac{W h\, D\, l}{2} \times \frac{1}{2} \times \frac{W h\, \pi D^2}{2\, b\, E}$$

$$= \frac{W^2 h^2\, \pi\, l\, D^3}{8\, b\, E} \text{ foot-pounds} \qquad (3)$$

Setting the expression (1) for the energy in the pipe equal to the two expressions (2) and (3) for the work done by the water in coming to rest, we get

$$\frac{W\, \pi\, l\, D^2\, V^2}{8\, g} = \frac{W^2 h^2\, \pi\, l\, D^2}{8\, K} + \frac{W^2 h^2\, \pi\, l\, D^3}{8\, b\, E},$$

and simplifying

$$h^2 = \frac{V^2}{W\, g \left(\dfrac{1}{K} + \dfrac{D}{b\, E} \right)} \qquad (4)$$

$$h = \sqrt{\frac{K}{W\, g}\, \frac{V^2}{\left(1 + \dfrac{K\, D}{E\, b} \right)}} \qquad (5)$$

Inserting the numerical value of $\sqrt{\frac{K}{W g}}$, this becomes

$$h = \frac{145\ V}{\sqrt{1 + \frac{K\ D}{E\ b}}} \quad \text{(A)}$$

Inserting values for steel pipe:

$$K = 4.2 \times 10^7$$
$$E = 4.0 \times 10^9$$

this becomes

$$h = \frac{145\ V}{\sqrt{1 + 0.01 \frac{D}{b}}} \quad \text{(6)}$$

Other Derivations.—Formula (A) has been derived by several other methods. One is as follows:

The velocity of a vibration along a pipe is shown to be

$$a = \sqrt{\frac{g}{W\left(\frac{1}{K} + \frac{D}{E\ b}\right)}} = \frac{4\,660}{\sqrt{1 + \frac{K\ D}{E\ b}}} \quad \text{(12)}$$

(See Formula (12) under the heading "Ordinary Water-Hammer".)

If the gate is closed instantly, the pressure will rise immediately to h at the gate, but it will take a time, $\frac{L}{a}$, before this rise has reached the upper end of the pipe, and before the last layer of water is brought to rest.

The principle of impulse and momentum is that the impulse (or force multiplied by time) of all forces acting on a collection of particles in any time is equal to the gain of momentum of the particles in that time (mass multiplied by velocity). In this case

$$-\ W h\ P \frac{L}{a} = -\ \frac{W\ P\ L}{g} V$$

$$h = \frac{V}{g} a \quad \text{(7)}$$

Inserting the value of a from Formula (12) we get, as before,

$$h = \frac{4\,660\ V}{g\sqrt{1 + \frac{K\,D}{E\ b}}} = \frac{145\ V}{\sqrt{1 + \frac{K\ D}{E\ b}}} \quad \text{(A)}$$

One point should be emphasized, namely, that as all the energy in the water is used in producing this maximum h, it is impossible to get a greater pressure than this. Therefore, any formula yielding greater results must be incorrect, as it violates the law of the conservation of energy.

What appears to the writer to be an incorrect use of the formula is given in one of the articles on this subject. This inserts the ordinary value of E_w for wood, and applies the formula to wood stave pipes.

If the staves are initially in compression, the term, Eb, should be replaced by $E_w\, b + E_s\, \phi$,

where E_w is the modulus of elasticity of the wood,
E_s " " " " " " " steel,
and ϕ " " cross-section of steel bands per unit of length of pipe.

If the staves are not initially in compression, the $E_w\, b$ drops out altogether. In any case, the sinking of the steel bands into the wood is neglected.

Ordinary Water-Hammer.

Derivation of Formula.—The maximum value of the water-hammer given by Formula (A) cannot occur if the reflected waves from the reservoir reach the gate before it is entirely closed. These waves return in what may be called the critical time, equal to $\frac{2\,L}{a}$, and, for cases where T is greater than $\frac{2\,L}{a}$, the maximum value will have to be less than that derived from Formula (A).

The rise of pressure with gradual closing is more difficult to determine than with instantaneous closing, but, as almost all cases come under this head, the problem is worthy of careful analysis.

When the gate is closed a slight amount, a wave is started up the pipe, and the pressure at the gate rises at once. If the gate is closed no farther, this pressure will continue constant until $T = \frac{2\,L}{a}$, at which time the reflected wave from the reservoir reaches the gate, and the pressure falls to normal. The pressure then falls below normal by an amount which would be identical with the above rise, if friction, etc., did not tend to damp out the vibrations. It then rises again, and repeats the cycle until entirely dissipated by friction.

If the gate continues closing, however, a new wave and corresponding rise of pressure will be started for each increment of the gate motion, and as long as the gate moves the pressure will continue to rise until the reflected waves interfere. It is assumed that the gate is closed in such a way that, in the absence of reflected waves,

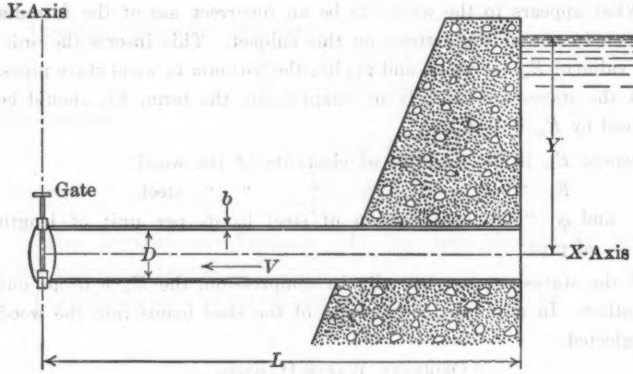

FIG. 1.

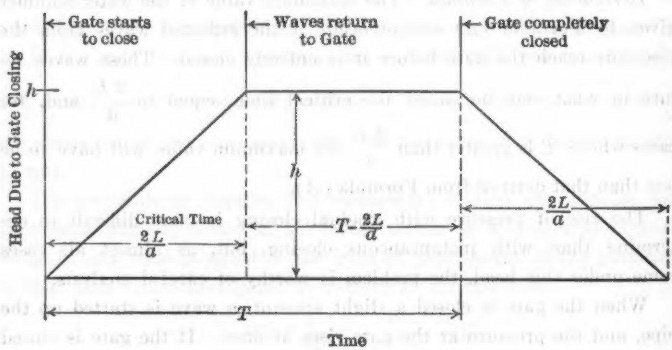

FIG. 2.

the pressure will rise at a constant rate. When the waves come back, the rate at which they are diminishing the pressure is the same as that at which the gate is increasing it. (See Fig. 2.) The pressure, therefore, will become constant until the gate is stopped, when it will begin to fall again, reaching normal after a time, $\frac{2L}{a}$. This is the

first assumption, and is very near to the truth, as experiments have shown.

In attempting to find a general formula, the principle of impulse and momentum was used. In its ordinary form, this principle is that the resultant impulse (force multiplied by time) or time integral of all the forces acting on any collection of matter during any time is equal to the gain of momentum (mass multiplied by velocity) of the matter in that time. In this case the application of this principle is complicated by the difficulty of accounting for the forces and velocity changes in the water that leaves the pipe. We may avoid this trouble, however, by considering the mass of water that just fills the pipe after the gate is closed. In an extreme case, with a velocity of, say, 12 ft. per sec. and a time of 3 sec., the lower end of this mass of water is about 18 ft. up the pipe at $t = 0$. As a good approximation, we may neglect this 18 ft. or less, in computing both the momentum of the water and the pressure at the end of this mass, which we take to be the same as at the gate. This is the second assumption. This would lead to errors for very high velocities or extremely slow closing, but not for ordinary cases.

According to the first assumption, stated previously, the pressure rises linearly from 0 to h in the time, $\frac{2L}{a}$, and then remains constant at h until the time, T, when it falls again. (See Fig. 2.) Therefore, the impulse (force multiplied by time) acting on this mass of water from the time, 0, to the time, T, is

$$-P\,W\frac{h}{2}\,\frac{2L}{a} - P\,Wh\left(T - \frac{2L}{a}\right) = -P\,Wh\left(T - \frac{L}{a}\right).$$

The assumption that the pressure, h, rises linearly to the time, $\frac{2L}{a}$, and then remains constant (h) until the time, T, or for a total time of $\left(T - \frac{2L}{a}\right)$, restricts the application of the formula to times greater than $\frac{2L}{a}$.

The velocity of the part in the pipe changes from V to 0*, and its

*A rigorous analysis from Allievi (given later) shows that, with the above assumptions, the velocity of the water at all points of the pipe is exactly 0 at $t = T$. This can be seen from the fact that, first, as the gate is closed, there is no velocity due to water flowing through the pipe; second, as the waves running up the pipe are exactly equal in intensity to the reflected waves coming down the pipe, there is no motion due to this cause.

mass, according to the second assumption, is $\frac{P\,W\,L}{g}$. The gain of momentum (mass multiplied by velocity), therefore, is $-\frac{P\,W\,L}{g}V$.

The equation of impulse and momentum, therefore, is

$$-P\,W\,h\left(T-\frac{L}{a}\right)=-\frac{P\,W\,L}{g}V, \text{ or}$$

$$h=\frac{L\,V}{g\left(T-\frac{L}{a}\right)}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(B)$$

This formula is simpler than Allievi's, and omits his most questionable assumption, but contains in its stead the second approximation noted previously, and, therefore, is subject to error for very slow closing or very high velocities. This formula does not give any value for the fall of pressure following the rise, nor does it apply for opening the gates.

For all ordinary cases it is as close as the assumptions and uncertainties of any actual case warrant, and its simplicity makes it preferable for common use. It is, moreover, the only formula which the writer has ever seen that is good for all values of T greater than $\frac{2\,L}{a}$, as one may see from the numerical cases given later.

For any time less than $\frac{2\,L}{a}$, as no reflected wave has returned to the gate before it is closed, the maximum pressure will be the same as if the gate was closed instantaneously (Formula (A)), no matter how the gate is closed. This follows mathematically from Formula (14) because $f\,(t)=0$ until $t=\frac{2\,L}{a}$.

Allievi's Formula.—A formula for ordinary water-hammer was derived along the following lines by L. Allievi,* with the assumptions that the area of the gate decreases linearly and the pipe velocity decreases at a constant rate throughout its closure.† It also assumes that T is not less than $\frac{2\,L}{a}$ and is good for such values of T only. The assumption of a linear rise of pressure, which is involved in that of a linear decrease of velocity, is confirmed by actual

* *Annali* della Societa degli Ingegneri, Vol. XVII, Rome, 1902.

† These two assumptions do not seem to be exactly consistent. The second alone is assumed in deriving Formula (B), and no assumption about the gate area is made there.

experiments under different conditions and different gate motions, and the formula itself has been checked for ordinary closing of gates where T is considerably greater than $\frac{2\,L}{a}$.

We assume the X-axis to be parallel with the pipe along the center line, starting from the gate, and plus in the direction of the reservoir. The sense of v is opposed to this being plus in the direction from the reservoir to the gate. The pipe is assumed to be horizontal, of constant diameter and thickness, and under pressure, but the formulas hold for inclined pipes as well. (See Fig. 1.)

The first fundamental equation is that the acceleration of any thin element of water of area, P, and thickness, $d\,x$, is caused by a difference of head on its two sides.

Therefore, as mass multiplied by acceleration equals force,

$$W\,P\left(y + \frac{\delta\,y}{\delta\,x}\,d\,x\right) - W\,P\,y = \frac{P\,W\,d\,x}{g}\,\frac{\delta\,v}{\delta\,t} \quad \text{(8)}$$

Simplifying, $\frac{\delta\,v}{\delta\,t} = g\,\frac{\delta\,y}{\delta\,x}$* .... (9)

which is the first fundamental equation.

The second equation results from the fact that there is a difference of velocity at any time at different points in the pipe in consequence of the deformations due to the rise in head, $d\,y$, which causes

First.—A compression of the water;

Second.—An enlargement of section due to stretching of the steel ring of the pipe which must be filled with water.

The first part of this is

$$\frac{d\,x}{K}\,W\,\frac{\delta\,y}{\delta\,t}\,d\,t \quad \text{(10)}$$

which is the distance along the pipe in which the water is compressed.

The lengthening of the pipe ring due to y is

$$\frac{W\,y}{E} \times \frac{D}{2\,b}\,\pi\,D = \frac{\pi\,D^2\,W\,y}{2\,E\,b} \text{ (See Formula (3).)}$$

The corresponding lengthening of diameter is

$$D_1 = \frac{D^2}{2\,E\,b}\,W\,y$$

*For inclined pipes, this becomes $\frac{\delta\,v}{\delta\,t} = g\,\frac{\delta\,y}{\delta\,x} + g$ sin. a, where a is the angle of inclination with the horizontal.

In a time, $d\,t$, this variation due to $d\,y$ is

$$d\,D_1 = \frac{D^2\,W}{2\,E\,b}\,\frac{\delta\,y}{\delta\,t}\,d\,t.$$

The corresponding enlargement of volume of an element of the pipe of length, $d\,x$, and area, $\frac{\pi\,D^2}{4}$, is

$$\frac{2\,\pi\,D\,d\,D_1\,d\,x}{4}\,; \quad (d\,D_1 = d\,D),$$

or, inserting the above value for $d\,D_1$,

$$\frac{\pi\,D^3\,W}{4\,E\,b}\,\frac{\delta\,y}{\delta\,t}\,d\,t\,d\,x$$

Dividing the volume by the area to get the difference in the distances along the pipe which the ends of the column have moved in taking up this new volume

$$\frac{D\,W}{E\,b}\,\frac{\delta\,y}{\delta\,t}\,dt\,dx \ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

which is the second part required.

The difference in velocities between any two points of the pipe is due to the sum of these two, or

$$(v + \frac{\delta\,v}{\delta\,x}\,dx)\,dt - v\,dt = \text{Formulas (10)} + \text{(11)},$$

or, inserting the values of Formulas (10) and (11) and simplifying,

$$\frac{\delta\,v}{\delta\,x} = \left(\frac{1}{K} + \frac{D}{E\,b}\right) W\,\frac{\delta\,y}{\delta\,t},$$

which is the second equation necessary.

Let
$$\frac{1}{a^2} = \frac{W}{g}\left(\frac{1}{K} + \frac{D}{E\,b}\right);*$$

$$a = \sqrt{\frac{g}{W\left(\frac{1}{K} + \frac{D}{E\,b}\right)}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

substituting this,

$$\frac{\delta\,v}{\delta\,x} = \frac{g}{a^2}\,\frac{\delta\,y}{\delta\,t} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

---

* a is really the velocity of the wave along the pipe, as is seen from Formulas (9*A*) or (14).

By elimination of v in Equations (9) and (13) we get the common wave equation,

$$\frac{\delta^2 y}{\delta x^2} = \frac{1}{a^2} \frac{\delta^2 y}{\delta t^2} \quad \text{(9A)}$$

likewise,

$$\frac{\delta^2 v}{\delta x^2} = \frac{1}{a^2} \frac{\delta^2 v}{\delta t^2} \quad \text{(13A)}$$

A general solution of Equations (9) and (13), or (9*A*) and (13*A*), is

$$\left.\begin{aligned} y &= Y + F\left(t - \frac{x}{a}\right) - f\left(t + \frac{x}{a}\right)^{*} \\ v &= V - \frac{g}{a}\left[F\left(t - \frac{x}{a}\right) + f\left(t + \frac{x}{a}\right)\right] \end{aligned}\right. \quad \text{(14)}$$

these being general expressions for waves traveling in both directions with velocity, a.

Introducing in the first part of Equation (14) the fact that when $x = L$, $y = Y$, for all values of t, we get

$$f\left(t + \frac{L}{a}\right) = F\left(t - \frac{L}{a}\right) \quad \text{(15)}$$

To get rid of the unknown function, $f\left(t + \frac{x}{a}\right)$, we may express it in terms of F. To do this, choose t_1 so that

$$\left.\begin{aligned} t_1 + \frac{L}{a} &= t + \frac{x}{a} \\ t_1 - \frac{L}{a} &= t + \frac{x}{a} - \frac{2L}{a} \end{aligned}\right\} \quad \text{(16)}$$

Therefore, from Equations (15) and (16),

$$f\left(t + \frac{x}{a}\right) = f\left(t_1 + \frac{L}{a}\right) = F\left(t_1 - \frac{L}{a}\right) = F\left(t + \frac{x}{a} - \frac{2L}{a}\right). \quad \text{(17)}$$

Formulas (17) and (14) give

$$\left.\begin{aligned} y &= Y + F\left(t - \frac{x}{a}\right) - F\left(t + \frac{x}{a} - \frac{2L}{a}\right) \\ v &= V - \frac{g}{a}\left[F\left(t - \frac{x}{a}\right) + F\left(t + \frac{x}{a} - \frac{2L}{a}\right)\right] \end{aligned}\right\} \quad \text{(18)}$$

By Taylor's theorem
$$\left\{\begin{aligned} F\left(t - \frac{x}{a}\right) &= F(t) - \frac{x}{a} F'(t) + \Sigma_1 \\ F\left(t + \frac{x}{a} - \frac{2L}{a}\right) &= F(t) - \frac{2L - x}{a} F'(t) + \Sigma_2 \end{aligned}\right\} \quad \text{(19)}$$

* For inclined pipes this would have a term, $- x \sin. \alpha$, which would disappear at $x = 0$, the gate.

in which Σ_1 and Σ_2 represent terms containing derivatives of $F(t)$ of higher order. If we now suppose Σ_1 and Σ_2 to be negligible,* substituting Equation (19) in Equation (18), we get

$$y = Y + 2\frac{L - x}{a}F'(t) \qquad (20)$$

$$v = V - \frac{2g}{a}\left[F(t) - \frac{L}{a}F'(t)\right].$$

We assume the rise of pressure to be at a constant rate until the wave returns, that is

$$F'(t) = \text{constant}.$$

Thus, by Equation (20), as y_0 and v_0 are the values of y and v, respectively, at $x = 0$ (the gate),

$$\frac{dy_0}{dt} = 0, \text{ and } \frac{d^2 v_0}{dt^2} = 0$$

at any time after the reflected wave returns. As the equation of flow (neglecting friction) is

$$u^2 = \frac{v_0^2}{[\psi(t)]^2} = 2g\,y_0,$$

and as $\frac{dy_0}{dt} = 0$ (the assumption of linearity),

$$\frac{dv_0}{dt} = \psi'(t)\sqrt{2g\,y_0}.$$

Differentiating the second part of Equation (20) for $x = 0$, $v = v_0$, we get

$$\frac{dv_0}{dt} = -\frac{2g}{a}F'(t).$$

Equating these two we get a value of $F'(t)$ which, substituted in the first part of Equation (20), gives a second degree equation in y_0 from which h, or $y_0 - Y$, can be obtained. Now, remembering that from linearity, $\psi'(t) = -\frac{V}{T\sqrt{2gY}}$,† and calling $N = \left(\frac{LV}{gTY}\right)^2$

$$\frac{y_0^2}{Y^2} - 2\frac{y_0}{Y}\left[1 + \frac{1}{2}\left(\frac{LV}{gTY}\right)^2\right] + 1 = 0 \qquad (21)$$

---

* Σ_1 is not negligible if $\frac{x}{a} > t$, as the wave has not yet arrived at the point. Likewise, Σ_2 is not negligible if $\frac{2L - x}{a} > t$, as the reflected wave has not yet got back there. Therefore, the following formulas are incorrect if the time of closing is less than $\frac{2L}{a}$. Mathematically, this follows, because F is defined by different algebraic expressions for positive and negative values of its argument. Thus, a Taylor's series for positive values will not hold for negative values.

† This brings in the assumption of linear gate closing, which is used here only. This is probably inconsistent with the other assumption of linear rise of pressure.

Solving this, and putting $h = y_0 - Y$, since $t > \frac{2\,L}{a}$,

$$\left.\begin{aligned} \frac{h}{Y} &= \frac{N}{2} \pm \sqrt{\frac{N^2}{4} + N} \\ h &= \frac{N\,Y}{2} \pm Y\sqrt{\frac{N^2}{4} + N} \end{aligned}\right\} \ldots\ldots\ldots\ldots\ldots (C)$$

As a matter of fact, this formula becomes inaccurate as it approaches $\frac{2\,L}{a}$. This discrepancy must be due to some error in the assumptions. The assumption that $\psi\ (t)$ and $F\ (t)$ are both linear functions seems to be most open to question.

Experiments have shown that $F\ (t)$ (the rise of pressure) is often linear. Therefore, the other assumption, namely, that $\psi\ (t)$ (rate of closing of gate), is at the same time linear is probably at fault. This latter assumption is not used in obtaining Formula (B).

The minus sign in Formula (C) is supposed to apply in opening the gates, and the drop of pressure, according to the formula, can never exceed Y, but approaches it as a limit.

Surge in Surge Tanks.

The problem of ordinary surge-tank design is simple, as far as the theory and fundamental assumptions are concerned. In starting up, the water is at rest in the entire system. Suddenly the load is thrown on, and, assuming that the full load flow is immediately drawn from the surge tank, a known quantity of water is drawn out, and the quantity of water coming in depends on the difference in head between the surge tank and the reservoir.

Neglecting friction, this problem is of the simplest, and can be derived at once from work and energy. The answer is the same, either for starting or stopping the plant.

When friction is considered, the fundamental equations are still simple to set up, but the solution becomes difficult, even with the aid of the higher mathematics. The following solution is offered by Dr. D. L. Webster, of the Physics Department of Harvard University:

To determine the motion of the water in the pipe, consider the work done on it and the kinetic energy it acquires in moving an

infinitesimal distance, dx. The amounts of work done by various forces are as follows:

(a) By gravity: $(W P\, dx)\, s$

(b) By friction: $(W P\, dx)\left(-f \frac{v^2}{V^2}\right)$,

f being the head loss by friction alone at the limiting velocity, V, and, therefore, $f \frac{v^2}{V^2}$ being the loss at the velocity, v. (Friction varies as velocity squared.)

The kinetic energy acquired by the water in the pipe at the same time is $\frac{W P L}{g} d\left(\frac{v^2}{2}\right) = \frac{W P L}{g} v\, dv$; and the energy acquired by the water entering the pipe is $W \frac{P\, dx}{g} \frac{v^2}{2}$.

Equating the work done to the energy acquired, we find

$$\left[s - f \frac{v^2}{V^2} - \frac{v^2}{2\,g}\right] dx = \frac{L}{g} v\, dv \qquad (21)$$

or, as $dx = v\, dt$,

$$\frac{d\,v}{dt} = \frac{g}{L}\left[s - \left(\frac{f}{V^2} + \frac{1}{2\,g}\right) v^2\right] \qquad (22)$$

This equation may be further simplified by noticing that, in a steady condition, s becomes the final head loss, F, by friction and entrance to the pipe together, and $\frac{d\,v}{dt}$ becomes zero and $v = V$, whence

$$F = f + \frac{V^2}{2\,g} \qquad (23)$$

and

$$\frac{d\,v}{dt} = \frac{g}{L}\left[s - F \frac{v^2}{V^2}\right] \qquad (24)$$

In this equation, v and s are both variables, dependent on t, so that, as s is the one in which we are interested, it would be well to express the equation in terms of t, s, $\frac{d\,s}{dt}$, and $\frac{d^2\,s}{dt^2}$. This we may do by using the equation

$$A\,s = Q\,t - P\,x \qquad (25)$$

derived by considering the volumes leaving and entering the standpipe. By differentiating Equation (25), we may obtain

$$A \frac{d\,s}{dt} = Q - P\,v, \quad A \frac{d^2 s}{dt^2} = -P \frac{d\,v}{dt} \qquad (26)$$

Therefore, substituting in Equation (24),

$$\frac{d^2 s}{dt^2} - \frac{g F}{P L A V^2}\left(Q - A \frac{d s}{dt}\right)^2 + \frac{g P}{L A} s = 0 \ldots\ldots (27)$$

As it is impossible to integrate this equation, we must make some approximation, preferably one that will make the maximum draw-down greater, rather than less, than its real value, and thus be on the safe side. Such an approximation may be obtained by considering the head loss by friction and entrance proportional to the velocity, rather than to its square, having the actual value, F, when $v = V$, and therefore, a greater value for all smaller velocities.

Thus we may replace $F \frac{v^2}{V^2}$ in Equation (24) by $F \frac{v}{V}$ and replace Equation (27) by

$$\frac{d^2 s}{dt^2} + \frac{g F}{L V} \frac{d s}{dt} + \frac{g P s}{L A} = \frac{g F Q}{L A V} = \frac{g P F}{L A} \ldots\ldots\ldots (28)$$

When a steady condition is reached, with $v = V$, $\frac{d s}{dt}$ and $\frac{d^2 s}{dt^2}$ will be zero, and we shall have s equal to F, as we have seen above, or as we can derive from Equation (28), remembering that $\frac{Q}{P} = V$. Letting $s' = s - F$, $2 k = \frac{g F}{L V}$, and $n^2 = \frac{g P}{L A}$, Equation (28) becomes

$$\frac{d^2 s'}{dt^2} + 2 k \frac{d s'}{dt} + n^2 s' = 0 \ldots\ldots\ldots\ldots\ldots\ldots (29)$$

This equation is the common form for damped harmonic vibrations, and is treated at length in many text books on the calculus, such as Osgood's, where the general solution is shown to be of the form

$$s' = C_1 e^{-k t} \cos. n' t + C_2 e^{-k t} \sin. n' t \ldots\ldots\ldots\ldots (30)$$

where $n' = \sqrt{n^2 - k^2}$, and C_1 and C_2 are arbitrary constants to be determined from the initial conditions of the problem.

For the case where full power is turned on, with the water in the conduit initially at rest, we have when

$$t = 0, s' = - F, \text{ and } \frac{d s'}{dt} = \frac{Q}{A} \ldots\ldots\ldots\ldots\ldots\ldots (31)$$

From these we may at once obtain

$$C_1 = - F, \text{ and } - C_1 k + C_2 n' = \frac{Q}{A} \ldots\ldots\ldots\ldots\ldots\ldots (32)$$

or

$$s = F\left[1 - e^{-k t}\left(\cos. n' t + \frac{k}{n'} \sin. n' t\right)\right] + \frac{Q}{A n'} e^{-k t} \sin. n' t \ldots\ldots\ldots\ldots\ldots\ldots (33)$$

To find the maximum draw-down, we have now merely to differentiate this and set the derivative equal to zero.

Thus we find

$$\frac{ds}{dt} = e^{-kt}\left[F \left\{ k \left(\cos. n' t + \frac{k}{n'} \sin. n' t\right) + n' \sin. n' t - k \cos. n' t \right\} + \frac{Q}{A}\left(\cos. n' t - \frac{k}{n'} \sin. n' t\right)\right]$$

$$= e^{-kt}\left[F \frac{n^2}{n'} \sin. n' t + \frac{Q}{A}\left(\cos. n' t - \frac{k}{n'} \sin. n' t\right)\right] = 0. \quad (34)$$

This expression equals zero when

$$\tan. n' t = -\frac{n'}{\frac{F A}{Q} n^2 - k} = \frac{-n'}{k}, \quad \cos. n' t = -\frac{k}{n},$$

$$\sin. n' t = +\frac{n'}{n}.$$

Let t_1 = this value of t, when s is at its maximum, S.

From Equation (33) at time, t_1, $s = S$, where

$$S = F \left\{ 1 - e^{-kt_1} \left(-\frac{k}{n} + \frac{k}{n} \right) \right\} + \frac{Q}{An'} e^{-kt_1} \frac{n'}{n},$$

or, $$S = F + \frac{Q}{An} e^{-kt_1} = F + Q \sqrt{\frac{L}{g A P}}\, e^{-kt_1}. \quad \ldots\ldots (35)$$

This can be put in simpler form, for

$$k t_1 = \frac{k}{n'}\left(\pi - \cos.^{-1} \frac{k}{n}\right) \quad \ldots\ldots (36)$$

and

$$\frac{k}{n} = \frac{F}{2 Q}\sqrt{\frac{A g P}{L}} = m, \text{ and } \frac{k}{n'} = \frac{m}{\sqrt{1 - m^2}} = m'.$$

Therefore, $k t_1 = m' (\pi - \cos.^{-1} m)$, and Equation (35) becomes

$$S = F + \frac{Q \sqrt{\frac{L}{g A P}}}{e^{m'(\pi - \cos.^{-1} m)}} = F \left[1 + \frac{1}{2 m e^{m'(\pi - \cos.^{-1} m)}} \right]. (G)$$

This is the best approximation obtainable by this method.

The time required for a full cycle is $\tau = \frac{2\pi}{n'}$ .... (36A)

where $n' = \sqrt{n^2 - k^2} = n \sqrt{1 - m^2} = \sqrt{\frac{g P}{L A}} \sqrt{1 - m^2}$.

Thus $\tau = 2\pi\sqrt{\frac{L\,A}{g\,P} + \frac{F^2 A^2}{4\ V^2\ P^2}}$, approximately.

The time from starting to maximum draw-down is, from Equation (36)

$$t_1 = \frac{1}{n'}(\pi - \cos.^{-1} m) > \frac{\tau}{4}.$$

A second best approximation for S may be obtained, as $\frac{k}{n}$ is small, by taking $n' = n$ and

$$\cos.^{-1}\frac{k}{n} \doteq \frac{\pi}{2} \text{ and } k\,t_1 = \frac{\pi\,k}{2\,n} \ldots\ldots\ldots\ldots\ldots\ldots (37)$$

This gives, for this rougher approximation,

$$S = F + V\sqrt{\frac{L\,P}{g\,A}}\,e^{-\frac{\pi m}{2}} = F + \frac{Q\sqrt{\frac{L}{A\,g\,P}}}{e^{\frac{\pi m}{2}}} = F\left[1 + \frac{1}{2\,m\,e^{\frac{\pi m}{2}}}\right] \cdot (F)$$

A very rough approximation obtained from Equation (G), neglecting the effect of friction in damping the vibrations, though not in determining the final position, is

$$S = F + Q\sqrt{\frac{L}{g\,A\,P}} = F\left[1 + \frac{1}{2\,m}\right] \ldots\ldots\ldots (E)$$

A point to be noticed about these approximations is that the best, expressed by Equation (G), should give an answer slightly above the true one, the next best, Equation (F), somewhat higher, and the third best, Equation (E), the highest of all.

Although the most important quantity in the design of a surge tank is the maximum draw-down when throwing on the full load, with the water in the conduit initially at rest, the rise when the full power is suddenly shut off is also important, and may be found as a by-product of the foregoing analysis.

For this problem, the principal change in the analysis is the dropping of the term, Qt, in Equation (25), and the terms, such as the right hand side of Equation (28), that result from it. At the same time, the initial conditions must be changed to

$$s = +F, \text{ and } \frac{ds}{dt} = -\frac{Q}{A} \ldots\ldots\ldots\ldots\ldots (31A)$$

at $t = 0$; so that s is determined exactly as s' was in the previous work, but with the sign reversed. The new algebraic minimum value

of s, therefore, will be equal to minus the above maximum of s', making the maximum rise above the position of static equilibrium,

$$Q \sqrt{\frac{L}{g A P}} e^{-k t_1} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (38)$$

exactly as in Equation (35).

Thus it appears that the rise of the water from its position of kinetic equilibrium is exactly like its previous drop from its position of static equilibrium, so that the maximum height above the static position is less than the maximum draw-down from it by the distance, F, between the equilibrium positions.

Neglecting friction entirely, we get from Equation (E)

$$S = V \sqrt{\frac{L P}{g A}} = Q \sqrt{\frac{L}{g A P}} \ldots\ldots\ldots\ldots\ldots\ldots (D)$$

which is true for either starting or stopping.

This simple form can be checked at once by work and energy, since the work done in the surge tank is $\frac{1}{2} S^2 W A$, and the work done in starting or stopping the water equals its kinetic energy, or $\frac{W P L V^2}{2 g}$; equating these

$$\frac{1}{2} W A S^2 = \frac{1}{2} W \frac{P L V^2}{g}$$

$$S = V \sqrt{\frac{L P}{g A}} = Q \sqrt{\frac{L}{g A P}} \text{ as before} \ldots\ldots\ldots\ldots (D)$$

Economical Size of Penstock.

In developments involving the use of long flow lines, or pipes under a high head, the cost of the pipe line is sometimes the largest part of the total cost. For this reason its most economical size becomes a matter of the greatest importance.

The following formula was first derived for a rough check on the independent calculations of several engineers in determining the best size for a certain pipe. The result of the somewhat academic mathematical treatment was so simple that it led the writer to suspect its practical value. It was not until after it had been checked repeatedly by other methods, starting with quite different assumptions, that it seemed sufficiently useful to present for general use.

One point especially should be emphasized, namely, that a wide variation in the assumptions will lead to a comparatively slight variation in result, and the first approximation, in which the assumptions may be fairly rough, will generally yield a result very close to the final figure. For instance, a variation of 100% in any one of the assumptions will make a difference of 1 ft., only, in a 10-ft. pipe.

***Derivation.*—By the Chezy formula, the head lost in a pipe is**

$$\frac{LV^2}{C^2R} \text{ feet} \quad (40)$$

where $R = \frac{D}{4}$.

The income lost from this loss of head is

$$\frac{LV^2}{C^2R} i \text{ dollars per year} \quad (41)$$

This is clear loss, as the operating and maintenance expenses of the plant are assumed to be fixed.

The other factor which must be considered is the annual fixed charge on the pipe. This is

$$\frac{p}{100} BLD \text{ dollars per year} \quad (42)$$

It is clear that the sum of these two factors is the total yearly loss, and a pipe having a diameter which will make this sum a minimum will be most economical. Adding these and inserting the value $R = \frac{D}{4}$ **and** $V = \frac{q}{\frac{\pi D^2}{4}}$, **the yearly loss is**

$$\frac{16 \times 4 \times L q^2 i}{C^2 D^4 \pi^2 D} + \frac{pBLD}{100} \quad (43)$$

or simplifying,

$$\frac{6.5\, L q^2 i}{C^2 D^5} + \frac{pBLD}{100} \quad (44)$$

In order to get the minimum value of this expression, with variation of *D*, it is merely necessary to differentiate once and set equal to zero.

$$0 = \frac{-5 \times 6.5 \times L q^2 i}{C^2 D^6} + \frac{pBL}{100} \quad (45)$$

$$\frac{3\,250\, L q^2 i}{C^2 D^6} = pBL \quad (46)$$

$$D = \sqrt[6]{\frac{3\,250\, q^2 i}{C^2 p B}} \quad (H)$$

Assumptions.—The foregoing formula depends for its accuracy on the exactness of the factors which enter into it, and although these may be varied considerably without materially affecting the results, nevertheless certain principles should be carefully noted.

Flow Through Penstock.—The most difficult factor to select in Equation (H) is q, the flow through the penstock.

In a plant with a single pipe line running at full load all its operating time, and under a constant head, the full load flow, Q, would of course be used. In other plants the probable operating conditions must be known and q selected accordingly. Certain generalities, however, apply to all cases.

Where there is more than one penstock, it is assumed that each one will carry a proportional part of the load throughout the year, even though one of them may seldom be used.

If a typical daily load curve can be assumed, the flow at each hour of the day can be obtained, and the square root of the average square may be used for q. The average is not used, for the loss of head varies as the velocity squared, and not as the velocity. The only time considered should be that in which the plant is in active operation, for when it is closed down, or when the wheels are merely turning over, there is no income being earned and the loss of head, if any, makes no difference.

As a matter of interest, it may be said that in several cases which were studied, with load factors varying from 10 to 40%, the value of q chosen was about 80% of the full load flow

Income per Foot of Head.—The income per foot of head is the total gross income of the plant divided by the average net head and the number of penstocks. It was calculated that a 10 000-kw. plant with four penstocks under a 100-ft. head would earn \$160 000 gross per year, or \$40 000 per unit. This gives \$400 per year per foot of head for each penstock, but for six-tenths of the year there was an excess of water and a loss of 2 or 3 ft. in the penstock could be offset by opening the gate and using more water at a sacrifice of efficiency. Therefore, only \$160 a year should be used for i in this case, as it is only during low-water periods that a larger penstock will earn additional income.

Percentage of Investment.—The percentage return on the investment which the initial cost of the penstocks is expected to give covers

fixed charges, depreciation, taxes, profit, and in fact everything. The profit should be the same as that which the total plant investment is expected to yield.

Cost of Penstock.—The formula assumes that the cost of the penstock in place varies as the diameter. This assumption is very close to the truth, within the narrow limits in which the penstock size may vary. For this variation the commercial thickness of plates or staves is the determining factor in pipe cost. The quantities of excavation and back-fill vary with the diameter squared, but it would not be proper to assume that the cost varied correspondingly, as the equipment, fixed charges, etc., form so large a part that a small percentage change in quantities would vary the cost more nearly as the diameter than as the square of the diameter.

For the first trial, a penstock size must be assumed before the cost can be estimated or the formula used. If the economical size differs greatly from this, a new value of B may have to be tried.

Summary of Formulas Proposed by Other Writers.

Maximum Water-Hammer.—All formulas for instantaneous closing agree, however they may be deduced. At first sight they may seem to differ, but by simplifying they all reduce to the same form. In several standard books, however, typographical errors and a confusion between feet and inches lead to erroneous results.

Ordinary Water-Hammer.—Several other formulas have been proposed—deduced from work and energy, or impulse and momentum, or from assumption and experiment.

The following formula is given by D. W. Mead, M. Am. Soc. C. E.,* as a general expression for water-hammer. It is also given by Mr. Uhl in his paper,† but he considers it less accurate than Allievi's formula.

Assuming a head, h, caused by the closing of the gate and acting throughout the time, T, by impulse and momentum

$$P\, W h\, T = \frac{W\, L\, P\, V}{g}$$

$$h = \frac{L\, V}{g\, T} \qquad (47)$$

* "Water Power Engineering," p. 450.

† *Transactions*, Am. Soc. Mech. Engrs., Vol. 34, p. 347.

This is erroneous, as it neglects the time, $\frac{2L}{a}$, required for the pressure wave to travel up the pipe before h becomes constant. This error is most important in small values of T, and for such values may yield results 50% too small.

This error is negligible, however, for larger values, and for such the formula is sufficiently accurate. It also neglects water entering and leaving the pipe, and thereby introduces a slight error for very large values of T.

This formula may be obtained at once as a special case of the general formula

$$h = \frac{L V}{g\left(T - \frac{L}{a}\right)} \qquad \text{(B)}$$

by neglecting $\frac{L}{a}$ when T is comparatively large. It is evidently 50% in error, however, when $T = \frac{2L}{a}$.

I. P. Church, Assoc. Am. Soc. C. E., quotes the following formula of Joukovsky,* and Mansfield Merriman,† M. Am. Soc. C. E., also gives it, but states that it is not applicable to all cases.

It is derived as follows: We have shown (Formula (7)) that the maximum water-hammer is $h = \frac{Va}{g}$. In this case the gate is closed instantaneously, or before any reflected waves have arrived to cut down the pressure. These arrive in the time, $\frac{2L}{a}$, so it has been erroneously assumed that, if the gate is closed in a longer time, the rise of pressure is proportionally less. Writing the proportion

$$h : \frac{Va}{g} :: \frac{2L}{a} : T,$$

$$h = \frac{Va}{g}\,\frac{2L}{aT} = \frac{2LV}{gT} \qquad (48)$$

This formula is wrong, theoretically, and does not agree with practice or experiment, except for the special case where $T = \frac{2L}{a}$, when it becomes the common formula, $h = \frac{Va}{g}$. It can be used for small values only, and for larger values may be almost 100% in error.

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* "Hydraulic Motors," p. 209.

† "Hydraulics," p. 391.

Surge in Surge Tanks.—Mr. R. D. Johnson has derived the following equation for maximum surge in surge tanks:

$$S = \sqrt{\frac{P\,L\,V^2}{A\,g} + F^2}$$

$$t_1 = \frac{\pi}{2}\sqrt{\frac{L\,A}{g\,P} + \frac{C^2\,V^2\,A^2}{P^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (49)$$

where $F = C\,V^2$.

The writer has derived, for an approximation:

$$S = \sqrt{\frac{P\,L\,V^2}{A\,g} + \frac{F^2}{9}} + \frac{1}{3}F \quad \ldots\ldots\ldots\ldots\ldots (50)$$

Mr. D. W. Mead has the following formulas:

Neglecting friction, but including governor action, S, for a drop in level is given by

$$S^2 - 2\,Y\,S = -\frac{2\,P}{A}\left(\frac{L\,V^2}{2\,g} + \frac{Y\,T\,V}{\pi}\right) \quad \ldots\ldots\ldots\ldots (51)$$

where
$$T = \pi\sqrt{\frac{A\,L}{P\,g}}.$$

To this F must be added to approximate friction. . . . . (52)

Including friction, Mr. Mead gives for upward surge:

$$S = \sqrt{\frac{P\,L\,V^2}{A\,g}} - \frac{F\,T\,Q}{3\,A} \quad \ldots\ldots\ldots\ldots\ldots\ldots (53)$$

His most exact formula is

$$S^2 - 2\,(Y - F)\,S = -\frac{2\,P}{A}\left(\frac{L\,V^2}{2\,g} - C\,V^3\,(0.37\,T_1) + \frac{Y\,T_1 V}{\pi}\right) \ldots (54)$$

Where $C = \dfrac{F}{V^2}$

$\dfrac{\tau}{2} = T = \pi\sqrt{\dfrac{L\,A}{P\,g}}$ (no friction or governor)

$G = \sqrt{\dfrac{P\,L\,V^2}{A\,g}}$ (as above)

J = answer to Mead's Equation (51) (no friction)

$T_1 = \dfrac{J}{G}\,T.$

Economical Size of Penstocks.—Using the same methods, a similar formula for economical penstock size can be developed, assuming that the pipe thickness varies as the diameter squared. This is sometimes theoretically true, but practically long lengths of the pipe are

made of constant thickness, and the allowance of metal for rust and water-hammer is independent of the head. Therefore, the foregoing formula is preferable, as it is more simple.

Numerical Examples.

Maximum Water-Hammer.

Formula (A).—A penstock, 10 ft. in diameter, ½ in. thick, and 400 ft. long, has water flowing in it at 6 ft. per sec. The gate is under a 100-ft. head. The maximum rise of pressure that can develop due to closing the gate is (Formula (A)):

$$D = 10$$
$$b = 0.041$$
$$L = 400$$
$$Y = 100$$
$$V = 6$$

$$\frac{D}{b} = \frac{10 \times 12}{\frac{1}{2}} = 240$$

$$h = \frac{145\ V}{\sqrt{1 + 0.01 \frac{D}{b}}} = \frac{145 \times 6}{\sqrt{1 + 0.01 \times 240}}$$

$$= \frac{870}{\sqrt{3.4}} = 465 \text{ ft.}$$

To this must be added the static head (100 ft.) making a total pressure at the gate of 565 ft., or 245 lb. per sq. in.

Ordinary Water-Hammer.

In order to test out and show the limitations of the four formulas for ordinary water-hammer, three cases are worked out:

Case I.—The foregoing data are used, with a closing time of 5 sec.

Case II.—The foregoing data are used, with a closing time of 0.32 sec.

Case III.—The foregoing data are used, with a closing time of 0.32 sec., but the head, Y, is taken as 400 ft., and not 100 ft.

Case I.

Using the foregoing data, with a closing time of 5 sec.

Formula (*B*).—By Formula (*B*),

$$h = \frac{L\,V}{g\left(T - \dfrac{L}{a}\right)}$$

where, for steel pipe, $a = \dfrac{4\,660}{\sqrt{1 + 0.01\dfrac{D}{b}}}$,

$$a = \frac{4\,660}{\sqrt{3.4}} = 2\,500,\ \frac{L}{a} = \frac{400}{2\,500} = 0.16,$$

$$h = \frac{400 \times 6}{32.2\ (5 - 0.16)} = 15.5 \text{ ft.}$$

Therefore, the total maximum head is 115.5 ft.

Formula (*C*).—From Formula (*C*),

$$h = \frac{N\,Y}{2} + Y\sqrt{\frac{N^2}{4} + N}$$

$$N = \left(\frac{L\,V}{g\,T\,Y}\right)^2 = \left(\frac{400 \times 6}{32.2 \times 5 \times 100}\right)^2 = 0.0222,$$

$$h = \frac{0.0222 \times 100}{2} + 100\sqrt{\frac{0.0222^2}{4} + 0.0222} = 16.1 \text{ ft.}$$

Formula (*47*).—Formula (47) gives

$$h = \frac{L\,V}{g\,T} = \frac{400 \times 6}{32.2 \times 5} = 15 \text{ ft.}$$

Formula (*48*).—Formula (48) gives twice this, or

$$h = \frac{2\,L\,V}{g\,T} = 30 \text{ ft.}$$

Case II.

Taking the same data, with a closing time of exactly $\dfrac{2\,L}{a} = 0.32$ sec., we know we should get the same results as from Formula (*A*), maximum water-hammer (465 ft.). Using the different formulas, we get the following:

$$\text{Formula } (B)\ h = \frac{400 \times 6}{32.2\ (0.32 - 0.16)} = 465$$

$$\text{“} \quad (C)\ N = 5.4,\ h = \frac{540}{2} + 100\sqrt{\frac{5.4^2}{4} + 5.4} = 630$$

$$\text{“} \quad (47)\ h = \frac{2\,400}{32.2 \times 0.32} = 232$$

$$\text{“} \quad (48)\ h = \frac{4\,800}{32.2 \times 0.32} = 465$$

Case III.

Taking Case II, with a head of 400 ft., the only formula which will be altered is Formula (C).

$$\text{With } Y = 400, N = 0.34, h = \frac{0.34 \times 400}{2} + 400\sqrt{\frac{0.34^2}{4} + 0.34} = 312.$$

Table 1 is a comparison of the foregoing.

TABLE 1.—COMPARISON OF RESULTS.

Case.	Value of T.	Value of Y.	Formula.	Head, in feet.	Percentage of difference.
I	5 sec.	100 ft.	(B) Correct formula	15.5	0
			(C) Allievi	16	+ 3%
			(47)	15	− 3%
			(48)	30	+ 94%
II	0.32 sec. $= \frac{2L}{a}$	100 ft.	(A) or (B) Correct formula	465	0
			(C) Allievi	630	+ 35%
			(47)	232	− 50%
			(48)	465	0
III	0.32 sec.	400 ft.	(A) or (B) Correct formula	465	0
			(C) Allievi	312	− 34%
			(47)	232	− 50%
			(48)	465	0

The comparison of numerical results in Table 1 for three different cases brings out the limitations of the various formulas very clearly, and establishes the following general rules:

Formula (B) is the best for general use.

Allievi (Formula (C)) is only good for comparatively slow closing times. For quick closing his formula is incorrect, and may give results too high or too low.

Formula (47) is useless for general application, but is a good check for slow closing of gates.

Formula (48) is useless for general application, but is a good check for quick closing of gates.

Surge in Surge Tanks.

Taking the following case: A pipe, 5 ft. in diameter, and 13 000 ft. long, is passing 112 cu. ft. of water per sec. at full load. There is a total loss of 20 ft. in the pipe running full. A surge tank 14 ft. in diameter is to be installed. The draw-down of the reservoir is 30 ft.

The average head on the wheels is 440 ft. Determine the minimum height of the surge tank required.

$Q = 112$;

$P =$ pipe area $= 19.6$ sq. ft.;

$L = 13\,000$ ft.;

$A =$ surge tank area $= 154$ sq. ft.;

$F = 20$ ft.;

$V =$ full load velocity $= \dfrac{112}{19.6} = 5.7$ ft. per sec.;

$Y = 440$ ft.

Insert these numerical values in the various formulas, and, solving for S, we get the results shown in Tables 2 and 3.

TABLE 2.—SURGE IN SURGE TANKS. RESULTS BY SEVERAL FORMULAS.

Conditions.	Formula.	Maximum surge, in feet, S.	Percentage of difference.
Neglecting friction..	(D) Common formula........................	41	0
	(51) D. W. Mead (First approximation including governor action)............	45	+7%
Including friction...	(G) D. L. Webster........................	46	0
	(49) R. D. Johnson........................	46	0
	(50) Writer's approximate formula........	48	+6%
	(F) D. L. Webster (First approximate)......	48	+6%
	(54) D. W. Mead (Most exact, friction and governor action)......................	50	10%
	(E) D. L. Webster (Second approximate)...	61	35%
	(52) D. W. Mead (First approximate).......	65	43%
Rise above reservoir level..............	(G) D. L. Webster........................	26	0
	(53) D. W. Mead........................	29	+12%

TABLE 3.—SURGE IN SURGE TANKS.

TIME FOR COMPLETE CYCLE; AND TIME TO MAXIMUM DRAW-DOWN (STARTING UP).

	Formula.	Time.	Percentage of difference from Formula (36).
Time for complete cycle.	(36A) D. L. Webster....................	$\tau = 365$	0
	(54) D. W. Mead (including friction)	$\tau = 390$	+7%
	(51) D. W. Mead (no friction)	$\tau = 354$	−3%
Time to maximum draw-down (starting up)........	(36) D. L. Webster....................	$t_1 = 102$ sec.	....
	(49) R. D. Johnson....................	$t_1 = 99$ sec.	....

From the results in Tables 2 and 3, using Webster's or Johnson's formula, the tank must rise 26 ft. above maximum reservoir level, and must extend 46 ft. below minimum reservoir level. Therefore, if the surge tank is 14 ft. in diameter, it must be $46 + 30 + 26 = 102$ ft. high.

Economical Penstock Size.

A development is to be built on a small stream by creating a very large reservoir which will completely equalize the flow of the stream; the water is to be conducted to the surge tank and penstocks by a single wood stave pipe, 13 000 ft. long. The head is 440 ft.; the average flow through the pipe while the plant is in operation is 112 cu. ft. per sec., which develops about 3 000 kw. There are two 2 000-kw. units. The plant is to be used as a base load plant, and the load is to be very nearly constant, therefore q may be assumed as 112 cu. ft. per sec.

The plant develops about 26 000 000 kw-hr. of primary power yearly, which, sold at 1 cent, will give a gross income of $260 000, or $590 per ft. of fall.

A velocity of 6 ft. per sec. is first assumed, which would require a 5-ft. pipe. Such a wood pipe, in place and back-filled, is estimated at $10 per ft., or $2 per ft. of diameter.

Inserting these values in the formula:

$q = 112$

$i = 590$

$C = 113$ (assumed constant for Chezy formula)

$p = 15\%$ (assumed for profit, maintenance, depreciation, etc.)

$B = \$2.$

$$d = \sqrt[6]{\frac{3\,250\, q^2\, i}{C^2\, p\, B}} = \sqrt[6]{\frac{3\,250 \times 112^2 \times 590}{113^2 \times 15 \times 2}}$$

$$= \sqrt[6]{6\,250} = 4.29 \text{ ft.}$$

A 4 ft. 3½-in. pipe is most economical, or, to make a more common size and err on the side of better regulation, say, a 4½-ft. pipe.

The writer wishes to acknowledge gratefully the help of Dr. D. L. Webster and the criticisms of George F. Swain, Past-President, Am. Soc. C. E., and Mr. J. F. Vaughan, in preparing this paper. The following books were also consulted.

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"Surges in Pipe Lines," by F. G. Baum, M. Am. Soc. C. E., *Engineering Record,* December 24th, 1910; with Discussion by Mr. M. L. Enger, *Engineering Record,* January 7th, 1911.

Of the foregoing English references, Simin gives the best discussion of maximum water-hammer, taken from Russian experiments. Allievi gives the only complete derivation of ordinary water-hammer formulas, and Johnson gives the only complete discussion of surge tanks.

DISCUSSION

Mr. Church.

IRVING P. CHURCH,* ASSOC. AM. SOC. C. E. (by letter).—This paper contains a valuable résumé of formulas useful to hydraulic engineers. Among those for the maximum surge in a surge tank, as due to a sudden full opening of the turbine or penstock gates, Equation (G) is referred to as "the most nearly accurate," this designation being adopted, it may be supposed, on account of the exact mathematical steps taken in its derivation by Dr. Webster; but, as this derivation† involves an assumption which is rather far from the truth, namely, that the friction head is proportional to the velocity, instead of the velocity squared, the propriety of such a designation may perhaps be questioned. It may be of interest, therefore, to compare results obtained from this formula with those based on the latter and more accurate relation between friction head and velocity.

The writer, therefore, has undertaken the solution of the numerical example treated on page 269, using the same method as in his discussion‡ of a paper by Mr. R. D. Johnson. This method involves plotting a curve from its differential equation and making use of the relation that friction head is proportional to the velocity squared. Some of the details and results will now be presented, with further deductions. The author's notation will be retained, with foot-pound-second units in the numerical work.

The data of this example will be taken as: $Q = 112$, $P = 19.6$, $L = 13\,000$, $A = 154$, $F = 20.5$, $V = 5.7$, and $Y = 440$, there being one slight deviation from the author's data, namely, 20.5 for F, instead of 20. The reason for this is that at first the writer took the phrase "head lost" (in the author's definition of F) too literally, as meaning the loss of head due to pipe friction plus that due to entrance resistance, on which basis the vertical distance of the water surface in the surge tank from that of the forebay during the final steady flow would be 20 ft. $+ \dfrac{V^2}{2g} = 20.5$ ft., which quantity entered into the writer's formulas. Later, a careful reading of other parts of the paper showed that the author intended his F to include the $\dfrac{V^2}{2g}$; but by that time so much tedious work had been done that the writer retained

* Ithaca, N. Y.

† Dr. Webster's derivation of Equation (G), as given in the paper, seems to have followed the same procedure as that pursued in Professor Prasil's article in the *Schweizerische Bauzeitung* of 1908 (Nov., Dec., pp. 271, 301, 317, and 333). In this article the assumption is made that friction head is proportional to the velocity in order to render integrable the differential equation involved. (Professor Prasil's article, as translated for *The Canadian Engineer,* and printed in that journal during August and September, 1914, may now be procured in pamphlet form.)

‡ *Journal,* Am. Soc. of Mech. Engrs., June and October, 1908.

the value of 20.5 for the author's F and, accordingly, solved Equation (G) with $F = 20.5$, to get Dr. Webster's result for S. Mr. Church.

If we write Dr. Webster's Equation (26) in the form

$$A \frac{d\,s}{d\,t} = P\,(V - v)$$

and eliminate $d\,t$ by the aid of Equation (24), there is obtained

$$s\,d\,s + \frac{F}{V^2} v^2\,d\,s = \frac{L\,P}{A\,g}\,[V\,d\,v - v\,d\,v] \qquad \text{(100)}$$

in which there are only two variables, s and v. Integrating each term, between the limits, 0 and S, for s, and the corresponding limits, 0 and V, for v (since, when $s = S$, the rate of flow, Pv, in the pipe is equal to Q, that is, v is then equal to V), we have

$$\frac{S^2}{2} + \frac{F}{V^2} \int_0^S v^2\,d\,s = \frac{L\,P}{A\,g}\left[V^2 - \frac{1}{2}\,V^2\right] \qquad \text{(101)}$$

that is,

$$S^2 + \frac{2\,F}{V^2} \int_0^S v^2\,d\,s = \frac{L\,P}{A\,g}\,V^2 \qquad \text{(102)}$$

Equation (102) embodies the relation that friction head is proportional to the velocity squared, which it is intended to retain in order to compare the results with Equation (G).

The integration, $\int_0^S v^2\,d\,s$, is only indicated, and could be rigorously carried out only by knowing v as a function of s, which function is as yet unknown.

In Fig. 3 let $O\,H\,K$ be the vertical axis of the surge tank, O being the position of the water surface in it (static level) at the beginning of the surge due to a sudden full opening of the turbine gates, and H the unknown position of this surface at the end of the first downward surge, that is, $\overline{OH} = S$.

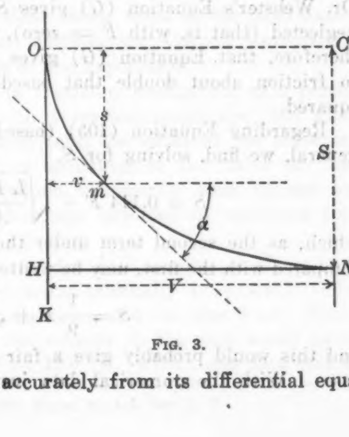

Fig. 3.

At any instant during the surge the water surface is some distance, s, below O, and the water in the main pipe has some velocity, v. If v be plotted as the horizontal co-ordinate and s as the vertical, with origin at O, the curve, $O\,m\,N$, is the result. In any numerical case this curve can be plotted quite accurately from its differential equa-

Mr. Church. tion, which we already have in Equation (100), and which may be thrown into the form,

$$\frac{d\,s}{d\,v} = \frac{L\,P}{A\,g} \cdot \frac{V - v}{s - \frac{F}{V^2}\,v^2} \qquad (103)$$

This gives the value of the "slope", or tan. α, for any point, m, of the curve (with due regard to scales).

Now, at the point, O (beginning of surge), both s and v are zero, so that tan. α = infinity, and hence the tangent line at O is vertical; and, at the end of the surge (point N), $v = \frac{Q}{P} = V$, and tan. α is zero, showing that the tangent line at N is horizontal (note that these positions of the terminal tangents are also found in the quadrant of an ellipse). Starting at O, then, we may plot the curve by its differential equation (successive tangents, with points of tangency). This was done by the writer, using co-ordinate paper of $\frac{1}{20}$-in. "mesh". On the drawing thus made, $\overline{OH}$ and $\overline{ON}$ were lengths of some 21 and 28 in., respectively.

By this curve a value of 43.25 ft. was found for S. From the curve there was also found, by numerical integration,

$$\int_0^S v^2\,d\,s = 0.1138\,V^2\,S \qquad (104)$$

which, substituted in Equation (102), gives the quadratic,

$$S^2 - 0.228\,F\,S = \frac{L\,P\,V^2}{A\,g} \qquad (105)$$

the solution of which gives the same value, 43.25, for S, which is a good check on the plotting of the curve. For the same numerical data, Dr. Webster's Equation (G) gives $S = 46.08$ ft.; and, if friction is neglected (that is, with F = zero), $S = 40.87$ ft. It would appear, therefore, that Equation (G) gives an increase in the surge as due to friction about double that based on the relation of the velocity squared.

Regarding Equation (105) (based on the plotted curve) as fairly general, we find, solving for S,

$$S = 0.114\,F + \sqrt{\frac{L\,P\,V^2}{A\,g} + 0.0130\,F^2} \qquad (106)$$

which, as the second term under the radical is generally quite small compared with the first, may be written

$$S = \frac{1}{9}\,F + \sqrt{\frac{L\,P\,V^2}{A\,g}} \qquad (107)$$

and this would probably give a fair approximation in most practical cases. With the numerical data just treated we find $S = 43.19$ ft.,

Mr. Church.

from Equation (107) [as compared with 43.25 ft., from Equation (106)].

Noting that a quarter ellipse, plotted to S and V as semi-axes, deviated at no point more than slightly from the true curve, the writer thought the former curve might be a fair substitute for the latter in all cases, so far as obtaining the value of the "frictional" integral, $\int_0^S v^2\, d\, s$, in terms of V^2 and S is concerned, which being done, Equation (102) becomes a quadratic easily solved for S.

The equation of the quarter ellipse being

$$S\, v = V\left[S - \sqrt{S^2 - s^2}\right] \ldots\ldots\ldots\ldots (108)$$

the summation, $\int_0^S v^2\, d\, s$, was found to be $\left(\frac{5}{3} - \frac{\pi}{2}\right) V^2 S$, and this, in Equation (102), gave

$$S^2 + 0.1917\, F\, S = \frac{L\, P\, V^2}{A\, g} \ldots\ldots\ldots\ldots (109)$$

and hence, finally,

$$S = 0.0958\, F + \sqrt{\frac{L\, P\, V^2}{A\, g} + 0.00918\, F^2} \ldots\ldots\ldots (110)$$

From this equation, with data of the numerical example just treated, we derive $S = 42.70$ ft.

Again, without notable error, this "elliptical" formula might be replaced by

$$S = \frac{1}{10}\, F + \sqrt{\frac{L\, P\, V^2}{A\, g}} \ldots\ldots\ldots\ldots\ldots (110)$$

It may be of interest to apply Equation (106) to the data of some experiments performed in 1910 by Mr. H. H. Conway and F. B. Storey, Jun. Am. Soc. C. E., in their thesis work at the Hydraulic Laboratory* of the College of Civil Engineering, Cornell University, and compare the results with those of the experiment.

A pipe, 140 ft. long and 4 in. in diameter, leading out of a pond, was furnished at the lower end with a vertical stand-pipe, and also a valve, opening into the atmosphere; and the effect of the more or less sudden or complete opening, or shutting, of the valve on the water column in the stand-pipe (which was hardly wide enough to be called a surge tank, perhaps, but was intended to represent the latter) was noted.

In one experiment, involving a sudden full opening of the valve, the water in pipe and stand-pipe being originally in a static condition, the stand-pipe was 6 in. in diameter, and the pipe 4 in. With steady flow, and valve fully open, the velocity in the pipe would have been $V = 7.5$ ft. per sec.; also, $F = 10.6$ ft. With these data and a

* In charge of E. W. Schoder, Assoc. M. Am. Soc. C. E.

Mr. Church. length of 140 ft., we obtain, from Equation (106), $S = 11.7$ ft. The experimental result was 11.6 ft.

Another experiment was made involving a sudden full opening of the valve, with the same data as before, except that the stand-pipe was 4 in. in diameter (that is, no wider than the main pipe), with $F = 10.8$ ft. and $V = 7.5$ ft. per sec. In this case, Equation (106) gives 16.93 ft. for S, as compared with 14.4 ft. by experiment. This rather considerable discrepancy may perhaps be explained by supposing that, on account of the small size of the stand-pipe, the assumption that it could deliver water at a constant rate to the valve during the whole time of surge was rather wide of the truth.

As another matter of interest in connection with Mr. Warren's paper, an experiment* in "ordinary water-hammer", made by Mr. B. F. Latting in 1894 at Cornell University, may be referred to. The pipe used was 8 in. in diameter, 2 395 ft. long, and was furnished at the discharging end with a nozzle 2 in. in diameter, provided with a valve gate, the static head being $Y = 302$ ft. There were two experiments, intended to be alike in conditions, in which the valve gate was gradually closed (after steady flow had once been started) and in such a way (so intended) as to produce a uniform negative acceleration (that is, retardation) in the velocity, v, in the pipe. The proper motion of the gate to produce this effect was investigated theoretically (see Equation (112)), but only gravity, friction, and the inertia of the water were brought into play, no account being taken of its elasticity, because the time of closing, $T = 25$ sec., was much greater than $\frac{2L}{a}$. The corresponding rate of decrease of gate area was not linear, with respect to the time, as Allievi's theory requires, but nearly so, with the important exception, however, that at the beginning of the closing an instantaneous movement of the gate was demanded through a small portion of its whole range. This, of course, was difficult to approximate to in the experimental handling of the gate. The initial velocity of the water in the pipe was $V = 6.4$ ft. per sec. The pressure at the lower end of the pipe (near the base of the nozzle) was observed at regular intervals.

Neglecting the elasticity of the water and the pipe, we have, at any instant of the closing, for the pressure (head) (above atmosphere) at the lower end of pipe,

$$y = Y - \frac{L}{g}\frac{dv}{dt} - \frac{F}{V^2}v^2 \ldots\ldots\ldots\ldots\ldots (111)$$

We have, also, at any instant, the relation:

$$y = \frac{v^2}{2g}\left[\left(\frac{P}{A'}\right)^2 + \zeta\left(\frac{P}{A_0'}\right)^2 - 1\right] \ldots\ldots\ldots\ldots (112)$$

* Described with related matters in the *Transactions*, Assoc. of Civ. Engrs. of Cornell University, for 1898, p. 31. The same article, "Unsteady Flow of Water in Pipes," was reprinted in the *Cornell Civil Engineer* of December, 1908, p. 61.

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in which A' = the variable gate area (opening), A_0' being its value at the beginning of the closing, while ζ is the coefficient of resistance of the nozzle (0.37). If an assumption is made for $\frac{dv}{dt}$ as a function of t, Equation (112) can be used to determine what function A' must be of t to produce the assumed relation for $\frac{dv}{dt}$.

If the retardation of the water in the pipe is to be uniform, we have $\frac{dv}{dt} = -\frac{V}{T}$, whence

$$y - Y = +\frac{L\,V}{g\,T} - \frac{F}{V^2}\,v^2 \ldots\ldots\ldots\ldots\ldots(112)$$

and hence, at the end of the closing, for the maximum $y - Y$, or h, we derive

$$h = \frac{L\,V}{g\,T} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(113)$$

For the data of Mr. Latting's experiment, this gives $h = 19.05$ ft., and Equation (B) of Mr. Warren's paper (with $\alpha = 4100$ ft. per sec.) gives 19.50 ft.; whereas, from Allievi's formula, Equation (C), we derive $h = 19.6$ ft. The experiment itself gave $h = 28.5$ ft.; but the gate was necessarily tardy in reaching the positions called for by theory near the beginning of the experiment, so that the retardation of the water was less at the beginning than intended by theory, and hence somewhat greater near the close.

Further details of the theoretical variation of pressure, velocity, area of gate opening, etc., during the closing, corresponding to various assumptions of the law of retardation of the velocity in a pipe, as a function of the time, may be found in the *Transactions* of the Association of Civil Engineers of Cornell University, already referred to. Numerical instances were also worked out, but the time, T, was so much greater than $\frac{2L}{a}$ that the compressibility of the water and distension of the pipe were not supposed to have any important effect on the results.

Mr. Johnson.

R. D. Johnson,* Esq.—The speaker was much interested in one point brought out by Mr. Warren, namely, that the dynamic pressure does not continue to rise throughout the entire operation of closing a gate. This statement on his part has led the speaker to refer to certain corroboratory testimony.

For slow-closing gates there would seem to be no important error introduced if the mass of water were treated as a rigid or non-elastic substance, especially if, in so doing, a mathematically exact statement

* New York City.

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could be written for the effect of the discharging water during the closure. An arithmetic solution (Table 4) is offered for the case where $L = 402$, $Y = 100$, $V = 6$, and $T = 2.5$, corresponding to Mr. Warren's assumptions, except as to the value of T.

TABLE 4.

Time (t).	Gate (G).	Velocity (v_0).	Total Head (y_0).
0	0.60	6.00	100
0.25	0.54	5.74	113
0.50	0.48	5.30	122
0.75	0.42	4.75	127.5
1.00	0.36	4.13	131
1.25	0.30	3.46	133.5
1.50	0.24	2.78	134.0
1.75	0.18	2.09	134.5
2.00	0.12	1.39	134.5
2.25	0.06	0.69	134.5
2.50	0.00	0.00	134.5

The values in Table 4 are intended merely for illustration, and are not particularly accurate. The length was chosen as 402, in order that the retardation for 0.25 sec. would be an even 0.02 of the dynamic back pressure, h. The time intervals may always be chosen in such a way as to make the retardation discernible by inspection of the instantaneous value of h, and this is essential to facilitate the balancing process of the arithmetical work. The first figure, 0.60, under the heading, "Gate", is a number representing full gate opening, and is chosen so that when it is multiplied by the square root of the total pressure head back of the gate, the product will equal the existing penstock velocity, thus, $G \sqrt{Y} = V$. Assuming uniform closure, the corresponding values of G are first written down opposite the time intervals. It is perhaps as well not to choose the time intervals shorter than $\frac{L}{a}$, because the whole mass cannot be affected within this time at any rate. A small error is introduced at the start, by this process, in assuming that the retardation for the first Δt is proportional to the dynamic pressure existing at the end of 0.25 sec., but this is smoothed out pretty well by subsequent computations. Of course, if $\Delta t = d t$, no error of this character would exist. It will be noticed by inspection that at any time $\Delta v_0 = 0.02 h$, and also, $G \sqrt{y_0} = v_0$, where G is for the time being regarded as a variable. This is the balance which must be effected by trial, but it is a very simple process if the relation between Δv_0 and h is made so straightforward as not to require any mental effort to convert them. Table 4 was worked out in a very few minutes with the aid of a 6-in. slide-rule. The speaker has worked out a large number of such computations in order to ascertain the power delivered to a water-wheel during gate motion,

open or shut, and thereby determine the speed variation when working on a given fly-wheel effect. The total head may also be varied, to account for any variation in the stand-pipe, and friction may also easily be included if desired. Mr. Johnson.

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In Fig. 4, let the circle, A, represent a large reservoir, from which extends a long, straight, and horizontal pipe, $A\ B$, of uniform section; and let reservoir and pipe be full of water.

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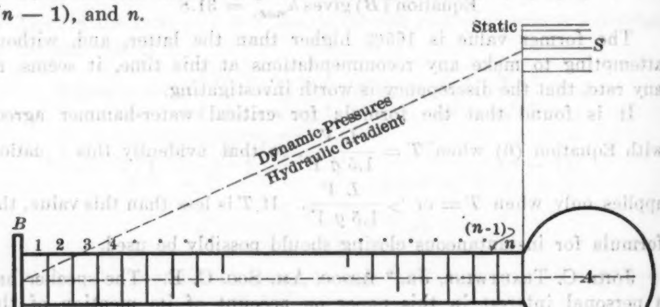
So long as the end, *B*, of the pipe remains open, the water flows from the reservoir, *A*, through the pipe, with velocity uniform as to both time and place, escaping freely at *B*. The pressure in the reservoir is always that due to the static head therein; but the pressures in the pipe (while the water escapes freely at *B*) decrease uniformly from a little less than static pressure, at a point, *n*, next the reservoir, to atmospheric pressure at the open pipe-end, *B*, as indicated by the hydraulic gradient. In other words, during flow, each cross-section of the pipe has its own proper dynamic pressure, depending on its distance from the origin, *n*. See the five diagrams at the top of Fig. 5; time = 0 sec.

Now let the gate at *B* be closed; and, for simplicity, let it first be supposed that the closure is instantaneous. Consider the water column, *A B*, as divided into *n* very thin and equal disks, 1, 2, 3, 4, . . . $(n-1)$, and *n*.



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The phenomena of water-hammer take place in a series of cycles, each consisting of four processes (and each process consisting of a conversion of kinetic into potential energy, or *vice versa*), as follows:

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During Process (1), a small quantity of water flows from the reservoir, *A*, into the pipe, occupying the space which otherwise would be left by the compression of the water and the extension of the pipe-walls.

Finally, at the instant when all the disks (including the small additional quantity mentioned) have been arrested, all the water in the pipe is at uniform superstatic or "high" pressure, and at rest; and a

wave of such pressure has traversed the length of the pipe, from gate to reservoir. **Mr. Trautwine.**

Fig. 5 traces the four processes of a first cycle in a pipe of such length that the pressure wave requires 4 sec. to traverse it in one direction from either end to the other. See the first and third, the third and fifth, etc., figures in the lower half of Fig. 5.

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The two solid arrows, next to the reservoir, between the first two figures, and between the second two figures, respectively, pointing from the reservoir, indicate the flow of water from reservoir to pipe during Process (1); the two dotted arrows near the middle of the pipe, with their plus signs, indicate the passage of the first pressure wave (a wave of increased pressure) through the pipe, from gate to reservoir.

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At the end of Process (2) (fifth figure, 8 sec.), all the water in the pipe is at uniform static pressure, but is in motion toward the reservoir; and

Mr. Johnson.

could be written for the effect of the discharging water during the closure. An arithmetic solution (Table 4) is offered for the case where $L = 402$, $Y = 100$, $V = 6$, and $T = 2.5$, corresponding to Mr. Warren's assumptions, except as to the value of T.

TABLE 4.

Time (t).	Gate (G).	Velocity (v_0).	Total Head (y_0).
0	0.60	6.00	100
0.25	0.54	5.74	113
0.50	0.48	5.30	122
0.75	0.42	4.75	127.5
1.00	0.36	4.13	131
1.25	0.30	3.46	133.5
1.50	0.24	2.78	134.0
1.75	0.18	2.09	134.5
2.00	0.12	1.39	134.5
2.25	0.06	0.69	134.5
2.50	0.00	0.00	134.5

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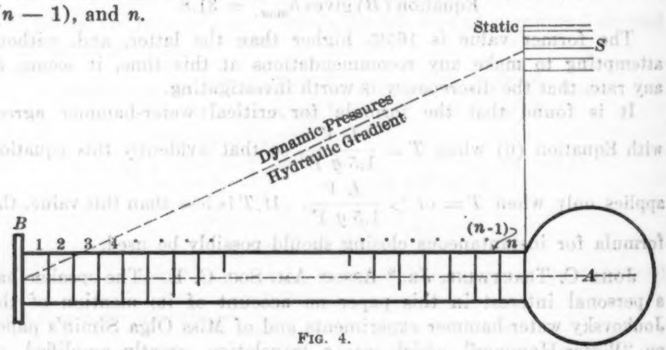

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Mr. Trautwine.

Process (3).—As there is no force to oppose this motion, it necessarily continues; and Disk 1, next to the gate, is left at a "low" pressure, theoretically as much below the static pressure as the static is below the "high" pressure; and, this taking place with each of the

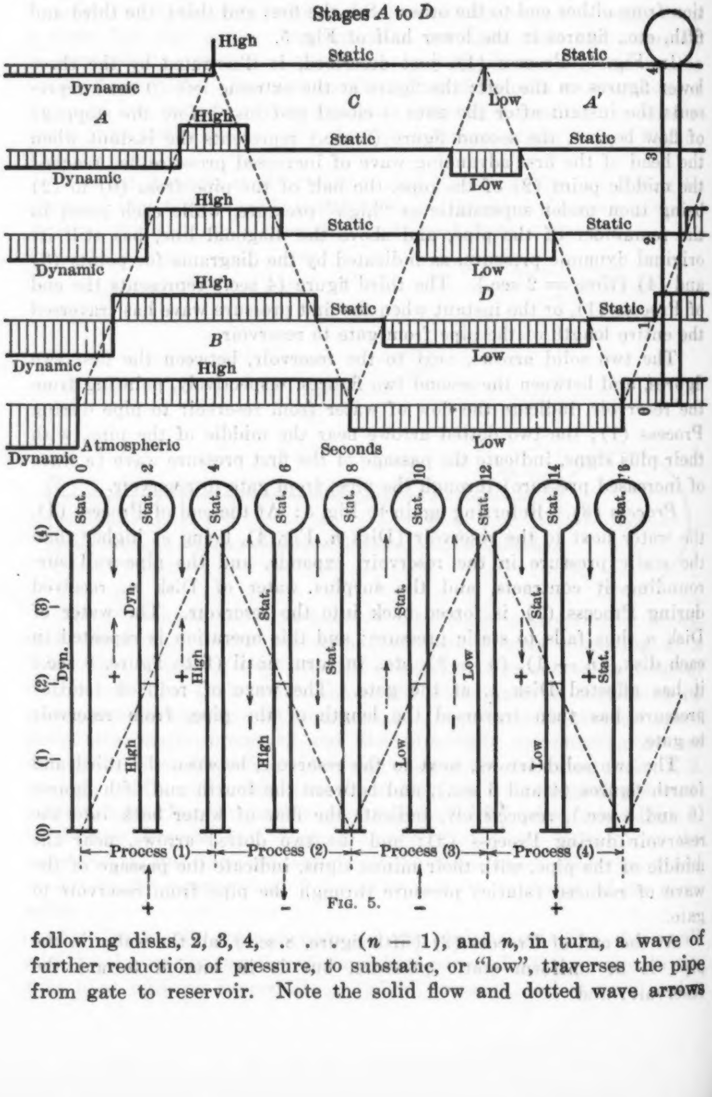

FIG. 5.

following disks, 2, 3, 4, ... $(n - 1)$, and n, in turn, a wave of further reduction of pressure, to substatic, or "low" traverses the pipe from gate to reservoir. Note the solid flow and dotted wave arrows

between the fifth and sixth figures (8 and 10 sec.) and between the sixth and seventh figures (10 and 12 sec.). Mr. Trautwine.

Process (4).—At the end of Process (3) (seventh figure, 12 sec.), the entire pipe sustains substatic or "low" pressure, and the water is at rest; but, the reservoir water being at a higher (static) pressure, flow again begins from the reservoir into the pipe, static pressure being thus re-established in Disks n, $(n - 1)$, . . . 4, 3, 2, 1, in turn, and a wave of increased pressure traversing the pipe from reservoir to gate. This is indicated by the arrows.

At the end of Process (4) (ninth figure, 16 sec.), the water is all flowing toward the gate, as in the figure on the extreme left; and, the velocity being the same as in that figure, the cycle of four processes is completed, although the pressures are now static and uniform, instead of dynamic and varying, as in the first figure.

The cycle is now repeated, and so on, until the system is brought to rest by the action of resistances; but, although the intensities of the pressures rapidly diminish, their durations remain constant through all repetitions until they cease entirely.

Summarizing: The direction and algebraic sense of the pressure wave and the direction of water flow, in each of the four processes of a cycle, are as given in the first four columns of Table 5, and as indicated by the dotted arrows and the signs (plus and minus) at the bottom of Fig. 5.

It will be noticed that when (Processes 2 and 3) the *pressure wave* is one of *diminution* (whatever its *direction*), the *water flow* is toward the *reservoir*, and *vice versa* in Processes 1 and 4.

The last two columns of Table 5 give the character of the energy (kinetic or potential) and of the pressure (dynamic, static, high, or low) existing throughout the whole length of the pipe at the beginning and at the end of each of the processes of the first cycle.

TABLE 5.

Process.	Wave. Direction toward:	Wave. Sense	Water flows toward:	Energy.	Nature of pressure in pipe.
				Kinetic......	Dynamic.
(1)...........	Reservoir.....	Plus	Gate..........		
				Potential.....	High.
(2)...........	Gate..........	Minus	Reservoir.....		
				Kinetic.......	Static.
(3)...........	Reservoir.....	Minus.........	Reservoir		
				Potential.....	Low.
(4)...........	Gate....	Plus	Gate..........		
				Kinetic.......	Static.
(1')...........	Reservoir.....	Plus	Gate..........		
				Potential.....	High.

Mr. Trautwine.

In the upper half of Fig. 5, the five influence diagrams, marked 0, 1, 2, 3, and 4, indicate, respectively, the histories of the corresponding selected points in the pipe; this history, for each point, consisting of the four stages, *A, B, C,* and *D*. Each stage is covered by the triangle enclosed between two of the dotted diagonal lines.

Thus, except at the gate, Point 0 (Disk 1, Fig. 4) and at the reservoir, Point 4 (Disk *n*, Fig. 4), the pressure, after the closing of the gate,

in stage	remains	until the point is reached by the head of the	wave of	pressure, in Process	The pressure then	and (see next line)
A	dynamic	advancing	increased	(1)	rises to high	
B	high	returning	reduced	(2)	drops to static	
C	static	advancing	further reduced	(3)	drops to low	
D	low	returning	increased	(4)	rises to static	
A′	static	advancing	further increased	(1′)	rises to high	
B′	high	returning	reduced	(2′)	drops to static	
C′, etc.						

Thus (the italics referring to the *horizontal* lines in the tabulation above), the pressure, in Stage *A*, remains *dynamic* until the point is reached by the head of the *advancing* wave of *increased* pressure, in Process (*1*). The pressure then *rises to high,* and (see next line) in Stage *B*, remains *high,* until . . . , etc., etc.

At the gate, Point 0 (Disk 1, Fig. 4), Stages *C* and *A′* (static pressure) are of zero duration; while, at the reservoir, Point 4 (Disk *n*. Fig. 4), Stages *B* and *D* (high and low pressure, respectively) are of zero duration.

In the foregoing it is assumed that the gate is completely closed. When the closure is only partial, the water, of course, continues flowing in the pipe; and, after such partial closure, the pressures, instead of being uniform (as in the case of complete closure), vary in intensity from a maximum, next to the reservoir, to a minimum, at the gate; and, for the datum, or "normal" pressure, to which the increases and diminutions of pressure are to be referred, we have, not the static pressure, as in the case of complete closure, but, at each point, the dynamic pressure proper to it by reason of its distance from the reservoir.

When (as in practice) the gate closure occupies a finite time, the vertical lines of the diagrams at the top of Fig. 5, are replaced by sloping lines, as in Fig. 6.

Analyzing the phenomena occurring in the case of such gradual closure, Mr. Boris Simin conceived of such closure as divided into a series of instantaneous partial closures, each having (for a given point in the pipe) its own separate influence diagram, analogous to those in Fig. 5, each such diagram having, of course, but a fraction of the total

height, and each following its predecessor, and superimposed above it, the series forming a composite diagram, approximating to a straight diagonal line, as the assumed partial closures are taken smaller and smaller. Mr. Trautwine.

It will be noticed that, in a given case, all points in the pipe are subjected to the same super-normal pressure, and to the same sub-normal pressure, independently of their distances from the reservoir; but that the several points sustain these pressures during periods of time ranging from zero (for the section at the reservoir) to the time

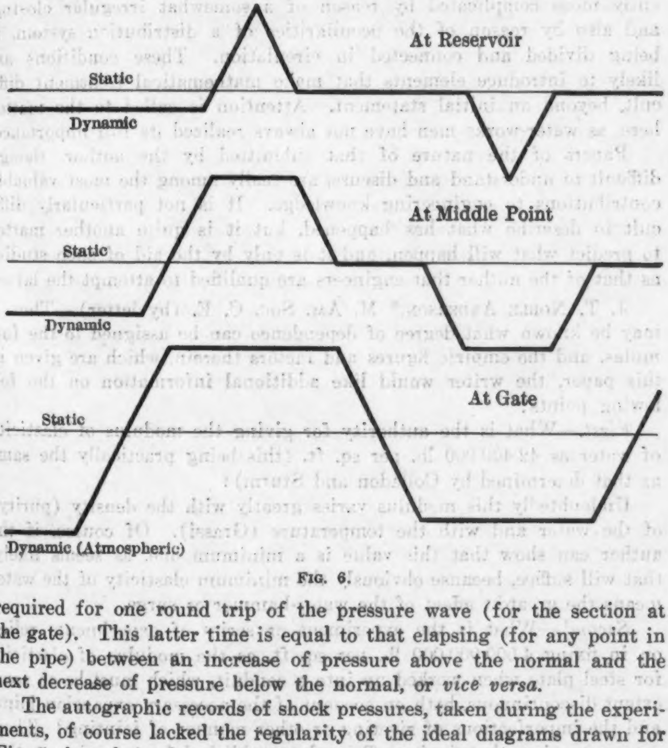

FIG. 6.

required for one round trip of the pressure wave (for the section at the gate). This latter time is equal to that elapsing (for any point in the pipe) between an increase of pressure above the normal and the next decrease of pressure below the normal, or *vice versa*.

The autographic records of shock pressures, taken during the experiments, of course lacked the regularity of the ideal diagrams drawn for Fig. 5; but their character indicated the correctness of the foregoing statement of the phenomena of water-hammer.

GARDNER S. WILLIAMS,* M. AM. SOC. C. E.—This problem is not solely a matter of pipes with which water-wheels are connected, nor Mr. Williams.

* Ann Arbor, Mich.

Mr. Williams.

is it one of the entire closing of gates. It is met frequently in the water distribution systems of manufacturing cities at about 5.30 or 6 P. M., the closing hour in large manufacturing establishments, when many of them, within a very few minutes, reduce their consumption of water; the resulting water-hammer in the pipes of the system becomes a very important matter, and, in several cases, has appeared to be the source of quite serious consequences.

It seems that the treatment of such a case is quite similar to that which has been presented by the author, except that it is apparently more complicated by reason of a somewhat irregular closing, and also by reason of the peculiarities of a distribution system, it being divided and connected in circulation. These conditions are likely to introduce elements that make mathematical treatment difficult, beyond an initial statement. Attention is called to the matter here, as water-works men have not always realized its full importance.

Papers of the nature of that submitted by the author, though difficult to understand and discuss, are really among the most valuable contributions to engineering knowledge. It is not particularly difficult to describe what has happened, but it is quite another matter to predict what will happen, and it is only by the aid of such studies as that of the author that engineers are qualified to attempt the latter.

Mr. Anderson.

J. T. NOBLE ANDERSON,* M. AM. SOC. C. E. (by letter).—That it may be known what degree of dependence can be assigned to the formulas, and the empiric figures and factors therein, which are given in this paper, the writer would like additional information on the following points:

First.—What is the authority for giving the modulus of elasticity of water as 42 400 000 lb. per sq. ft. (this being practically the same as that determined by Colladon and Sturm)?

Undoubtedly this modulus varies greatly with the density (purity) of the water and with the temperature (Grassi). Of course, if the author can show that this value is a minimum one, as seems likely, that will suffice, because obviously the minimum elasticity of the water means the greatest effect of the water-hammer or surge.

Second.—What is the experiment or series of experiments relied on in fixing 4 000 000 000 lb. per sq. ft. as the modulus of elasticity for steel plate when worked up into a conduit, which must be to some extent discontinuous, both on account of the necessary expansion joints and the imperfections of riveting or other manner of jointing? Then, even assuming that it is sufficiently established by experiment that these apparently disturbing elements make no appreciable difference (as has been claimed repeatedly), is not this modulus rather too low for modern steel plates. Obviously, for the same reason that it is safe

* Narbethong, Victoria, Australia.

to use the lowest modulus for the water, it is only safe to use the highest possible modulus for the steel. **Mr. Anderson.**

In the case of wood, it would seem likely that a double formula is required, because the thickness of the walls of the conduit—greater than a certain critical thickness—would entirely damp out the elasticity of the steel bands as a determining factor.

One other question, based on the writer's experience in pumping through long conduits: Does Fig. 2 represent, at all accurately, what actually happens? Will not the regularity of the recurrence of pulsations be disturbed by numbers of minor harmonic waves?

These minor harmonic waves are of more than academic interest, as the smallest appreciable disturbance in the load may often cause financial loss.

In conclusion, the writer notes that most of the symbols used by the author are in accord with those of well-known hydraulic writers, but one or two are disturbingly unfamiliar, namely, $a =$ velocity of vibration along pipe, in feet per second; and e (instead of Greek ϵ) $= 2.718$, base of Napierian logarithms; and he would like to emphasize the desirability of adopting a universal nomenclature and set of symbols for the whole hydraulic engineering world.

In 1908 the writer urged the introduction in Australia of the name "cusec", which had long been used in Anglo-India for the now universal unit of flowing water, the cubic foot per second; and the acre-foot, then used in America, as the most convenient unit for large irrigation storage reservoirs. It is gratifying to know that since that time these terms have been generally adopted in the irrigation parts of Australia.

H. C. VENSANO,* M. AM. SOC. C. E. (by letter).—The writer has read this paper with great interest, but disagrees with one of the author's assumptions and its resulting formula, and calls attention to a possible improvement in his formula for the economical size of penstock. **Mr. Vensano.**

Considering this latter first, attention is called to the fact that his formula, $D = \sqrt[6]{\dfrac{3\,250\, q^2\, i}{C^2\, p\, B}}$, includes B which, as defined by him, is "Cost of pipe in place, per foot of diameter per foot of length", is in itself a variable in D. The cost of metal conduits per foot of diameter is dependent on the weight of metal in the pipe. This depends on and is directly proportional to the thickness of the pipe, which in itself is directly proportional to the diameter. (If wood stave pipe is being considered, that portion of the cost represented by the bands will follow the same law.) In general, then, B is directly proportional to D, and Equation 44 can be written with its second

* San Francisco, Cal.

Mr. Vensano.

term as a function of D^2, bringing the final expression to the form, $D = \sqrt[7]{\dfrac{3\,250\, q^2\, i}{C^2\, p\; 2\, Z}}$, where constants are as assumed by Mr. Warren, and where B has been taken equal to $Z\, D$; Z being a constant.

This formula in the past has been proved in a little different form in connection with the discussion of a paper by the late Arthur L. Adams, M. Am. Soc. C. E., entitled "A Solution of the Problem of Determining the Economic Size of Pipe for High-Pressure Water-Power Installation".* In this paper Mr. Adams arrived at the theorem that:

"That pipe fulfills the requirements of greatest economy wherein the value of the energy annually lost in frictional resistance equals four-tenths (0.4) of the annual cost of the pipe line."

In the discussion of this paper by the late James H. Wise, Assoc. M. Am. Soc. C. E., and by Charles D. Marx, President, Am. Soc. C. E., and C. B. Wing, M. Am. Soc. C. E., this theorem was proved mathematically, and the resulting formulas can easily be changed to a form containing the seventh root of D. The formula in such form is very practical when considered with metal pipes, in that a pound price for "metal in place" may be used in it, rather than a price "per foot of pipe per foot of diameter." The price of metal per pound can be assumed closely for any given installation, whereas the price per foot of diameter, as already pointed out, varies with the diameter to be determined, and therefore is uncertain and difficult to fix.

Referring now to the formula developed for ordinary water-hammer, Mr. Warren writes:

"If the gate continues closing, however, a new wave and corresponding rise of pressure will be started for each increment of the gate motion, and as long as the gate moves the pressure will continue to rise until the reflected waves interfere. It is assumed that the gate is closed in such a way that, in the absence of reflected waves, the pressure will rise at a constant rate. When the waves come back, the rate at which they are diminishing the pressure is the same as that at which the gate is increasing it. (See Fig. 2.) The pressure, therefore, will become constant until the gate is stopped, when it will begin to fall again, reaching normal after a time, $\dfrac{2\,L}{a}$. This is the first assumption, and is very near to the truth, as experiments have shown."

Although the first two sentences of this assumption are no doubt allowable, and in fact in general are made by most investigators of this subject, the rest would not follow. Instead of the pressure rising steadily to a maximum and becoming constant until the gate is

* *Transactions*, Am. Soc. C. E., Vol. LIX, p. 173.

stopped, it should, following out the assumption of partial pressure wave returns, rise in a series of waves, reaching a definite maximum and then again falling to normal, but never passing to sub-normal until the gate is completely closed. The reason for this is that when a partial wave returns to the gate it not only returns to normal, thus neutralizing its own increment of pressure, but passes through normal to sub-normal, and, during the time of its second traverse of the pipe, counteracts in addition the new pressure increment, which is just leaving the gate, thus producing a gradual return of the total pressure in the pipe at the gate to normal. Mr. Vensano.

On the completion of one entire cycle in time, $\frac{4L}{a}$, each partial pressure wave again passes through normal to super-normal, and the total pressure in the pipe is gradually again brought up to its full value.

On such hypothesis, the water-hammer pressure at the gate during the time of gate closure (always provided the time of gate closure is longer than $\frac{4L}{a}$), would rise to a maximum a number of times, but would never, however, as stated previously, pass below normal until after the gate is completely closed. At a later time, of course, it would pass from super-normal to sub-normal in cycles, gradually dying out as the pipe friction reduced the wave effects.

To illustrate this, the writer submits Fig. 7, a diagram made up along the lines of those of Joukovsky, or at least as used by O. Simin in the translation of his work. Each line represents a partial pressure wave due to a partial gate closure. The point of start of each such line is, of course, at normal pressure. The stepped line at the bottom of the diagram gives the result of superimposition of all the partial pressure lines above. This diagram is made up for an assumed time of gate closure of 40 sec. and a length of pipe line (quite long) such that $\frac{2L}{a} = 10$ sec. It has only been drawn for the 40-sec. period, or the time of duration of gate closure, and has not been carried to the point where the waves would pass below normal. These waves are for a point indefinitely close to or at the gate. The sloping line, *A B C D E*, would be the resulting pressure line if infinitesimal pressure increments had been used.

Mr. Warren's diagrams showing a constant rise during the time interval, $\frac{2L}{a}$, a constant pressure during the time interval, $\frac{T-2L}{a}$, and a drop in pressure during the final period, $\frac{2L}{a}$, is therefore incorrect, which would apparently make his resulting formula likewise incorrect.

Mr. Vensano.

In substantiation of the contention that the assumption of constant pressure during gate opening is incorrect, the writer submits the following: The Pacific Gas and Electric Company, with which the writer is at present connected, completed in 1913 its Drum Hydro-Electric Plant. In connection with this station, a riveted penstock line about 6 300 ft. long and varying from $1\frac{1}{4}$ in. in thickness and 52 in. in diameter at the power-house to 72 in. in diameter and $\frac{1}{4}$ in. in thickness at its upper end, was built. The work was completed in November, and the writer attempted to make some tests on the pressure effects in the penstock due to opening and closing the power-house nozzles.

The flow of water at Drum is controlled by motor-operated needle-nozzles, which give an approximate linear decrease and increase of velocity of flow in the pipe. The time of closure was 75 sec.

Unfortunately, the weather was very cold at the time, and conditions were such as to interfere with the gauges by freezing. On this account, the results were rather unsatisfactory, and the tests were at that time abandoned. As the plant, since then, has been in operation continuously, it has not been possible to do anything further. It is hoped that these tests may be resumed in the near future.

The results obtained, however, showed that during the time of opening or closing the nozzles there were a number of rises and falls of pressure, and that the pressure did not remain constant during the entire period of operating the gate. The endeavor was made to take pressure readings at three points varying from 3 000 to 6 000 ft. from the power-house, while one or more nozzles in the station were opened and closed, both separately and together. Unfortunately, all the gauges, except one about 4 000 ft. from the station, were in exposed positions, and, because of the freezing of these two gauges, the corresponding results for different points on the line could not be obtained.

The writer suggests the general formula,

$$h = \frac{2\,l\,V}{g\,T} \quad \text{.........................(1)}$$

for the maximum pressure at any point in the pipe line and for any time of gate closure, with the limitation, however, that h can never be greater than $\frac{V\,a}{g}$, and that, when the formula gives greater values, h will equal the limiting value, $\frac{V\,a}{g}$. In this formula, h is the excess pressure over normal (not static) due to water-hammer, and l is the length of pipe, measured from the reservoir or source of supply to the point at which the pressure is desired. All other symbols are used as defined by Mr. Warren.

Mr. Vensano.

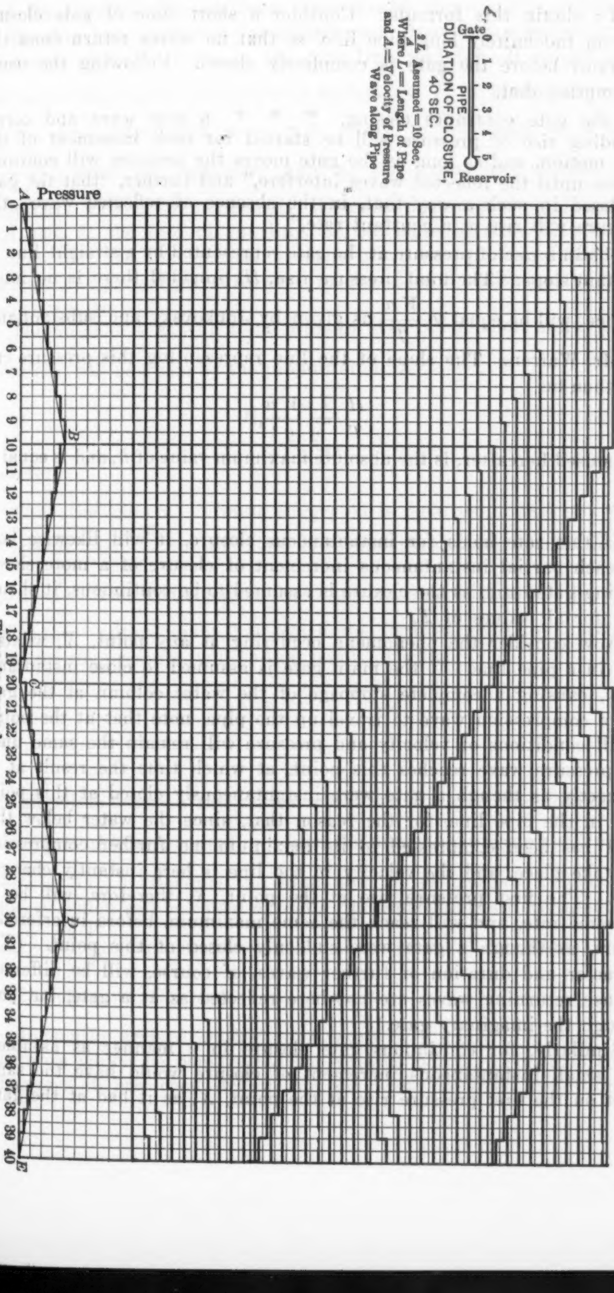

FIG. 7.

Mr. Vensano.

To obtain this formula: Consider a short time of gate closure and an indefinitely long pipe line, so that no waves return from the reservoir before the gate is completely closed. Following the usual assumption that,

"If the gate continues closing, * * * a new wave and corresponding rise of pressure will be started for each increment of the gate motion, and as long as the gate moves the pressure will continue to rise until the reflected waves interfere," and further, "that the gate is closed in such a way that, in the absence of reflected waves, the pressure will rise at a constant rate,"

we obtain a rise of pressure at the gate represented by a straight line of constant slope. The total pressure rise, H, provided there is no interference, will be equal to $\frac{V\,a}{g}$, as given by Joukovsky and substantiated by Mr. Warren. The slope of the line representing this pressure rise will then be

$$\frac{H}{T} = \frac{V\,a}{g\,t},$$

H, as used hereafter, is the absolute maximum value of h, and is equal to $\frac{V\,a}{g}$.

If this rate holds for instantaneous closure, it will likewise hold for each partial instantaneous increment of closure of a more slowly moving gate; and, as the closure is assumed to be continuous, it should hold for all values of T.

Now consider the phenomena occurring at any point, X, whatsoever in a pipe line. If the main gate is assumed to close instantaneously, causing a complete stoppage of the water column at the gate and a compression wave to travel up the pipe, note that at the point, X, all conditions of velocity and pressure will remain the same until the pressure wave reaches this point, at which time the result at X is exactly as though a gate were instantaneously closed at this point (X) in the pipe line, for the reason that, since the water below this point has been compressed to its maximum, no further compression can take place, and the velocity in the line is here instantly changed to 0. The first pressure rise at the point, X, therefore, will be at the same rate, and will reach the same maximum, before interference occurs, as though a gate were suddenly closed at this point. (The character and duration of further waves, of course, will be different, because returning waves are not here reflected as at a gate, but pass through the imaginary gate.)

Similarly, for continuous gate closure in a manner as assumed, each small instantaneous increment of closure would have the same effect on the first pressure rise at the point, X, as it had at the gate,

and the total of all such increments would likewise give results as at the gate until such time as interference from return waves occurs. Therefore, for any point in the pipe line, and for all values of T, the rate of first rise in pressure may be taken at $\frac{H}{T}$. Then at any point, X, the first interfering wave will return to this point at a time, $t = \frac{2\,l}{a}$, after the imaginary gate at this point has closed. The imaginary gate at X necessarily closes in the same time, T, as the real gate at the foot of the line, as the lapse of time between the instant that the first partial pressure wave passes X and the time that the last partial pressure wave reaches X and builds up the maximum pressure is T. Mr. Vensano.

The value of h at which the pressure rise is cut short at any point, therefore, will be given by the proportion $h : H :: \frac{2\,l}{a} : T$

or
$$H = \frac{a\,V}{g},$$

therefore,
$$h = \frac{2\,l V}{g\,T}.$$

Thus the general formula suggested is obtained. As h can never be greater than $\frac{a\,V}{g}$, for all values of l which give greater results, h must equal $\frac{a\,V}{g}$. The first (nearest the origin or reservoir) point, hereafter called the "critical point," at which the formula gives the maximum value for the pressure, will be that point for which

$$h = \frac{a\,V}{g} = \frac{2\,l\,V}{g\,T},$$

or
$$l = \frac{a\,T}{2}.$$

For all points between this critical point and the gate, the formula will give values of h in excess of $\frac{a\,V}{g}$, and the maximum pressure will obtain, (See Fig. 9 (c).) Between the origin and the critical point, the co-ordinates of which will be called h_y and l_y, the equation is strictly applicable, and $\frac{h}{l} = \frac{2\,V}{g\,T}$, which is a constant for any fixed set of conditions.

Therefore we may write $\frac{h}{l} = \frac{h_x}{l_x} = \frac{h_y}{l_y}$.

Joukovsky arrived at this same result by a study of pressure-wave diagrams which he constructed for various points. Fig. 8 is such a diagram; it has been drawn for the same assumed conditions as

Mr. Vensano.

Fig. 7, and shows graphically the pressure waves during the time of gate closure at a point three-fifths of the length of the pipe from the gate, or two-fifths of the distance from the reservoir.

By comparison of this diagram with Fig. 7, it can be seen that, for the case here represented, the maximum pressure, P', at this point is equal to 0.4 of the maximum pressure, P, at the gate, or in the ratio of $\frac{2}{5} L$ to L. From such study he formulated the conclusion that the ratio of the pressure at any point to the maximum pressure, is equal to the ratio of the time required for the wave to make a trip from the point in question to the reservoir and return to such point, to the time required to make a trip from the last (first where measured from the origin or reservoir) point of maximum pressure to the reservoir and return to such point. This conclusion he expressed by the formula,

$$\frac{P_x}{P} = \frac{t_x}{t},$$

which may be easily changed to the form,

$$\frac{h}{l} = \frac{h_y}{l_y}, \text{ as obtained previously.}$$

To show the general applicability of the proposed formula, the diagrams, Fig. 9 (*a*), (*b*), and (*c*), have been drawn for the three conditions, respectively,

$$T < \frac{2\,L}{a},$$

$$T = \frac{2\,L}{a},$$

$$\text{and } T > \frac{2\,L}{a}.$$

It is to be noted that, for times of gate closure greater than $\frac{2\,L}{a}$, the critical point is an imaginary one, and occurs on an imaginary extension of the pipe line beyond the gate.

Case I.—Instantaneous Closure.—For this case, $T = 0$.

$$h = \frac{2\,l\,V}{g\,T} \dots\dots\dots\dots\dots\dots\dots\dots (1)$$

substituting $T = 0$, $h = \infty$. This holds for all points in the line, but, by limitation, h cannot be greater than $\frac{a\,V}{g}$. Therefore

$$h = \frac{a\,V}{g}$$

for all points of the pipe, as it should for instantaneous closure.

Mr. Vensano.

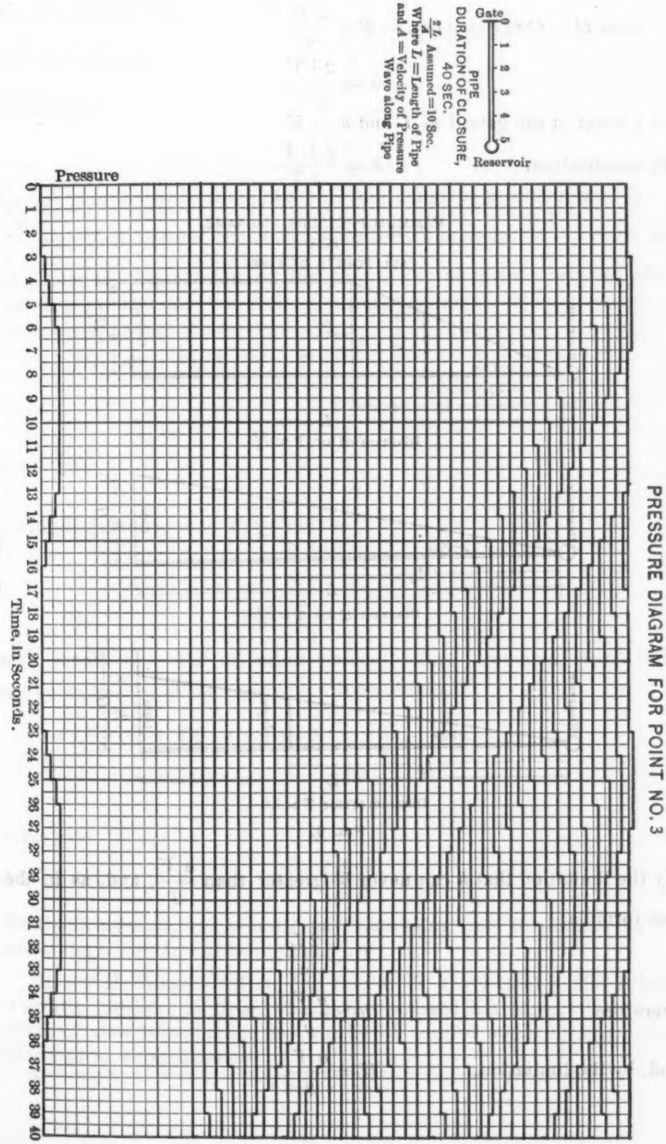

Fig. 8.

Mr. Vensano.

Case II.—(See Fig. 9 (a).)— $T < \frac{2L}{a}$

$$h = \frac{2 l V}{g t},$$

for a point at the gate, $l = L$ and $h = H$.

By substitution, $$h = \frac{2 L V}{g T}.$$

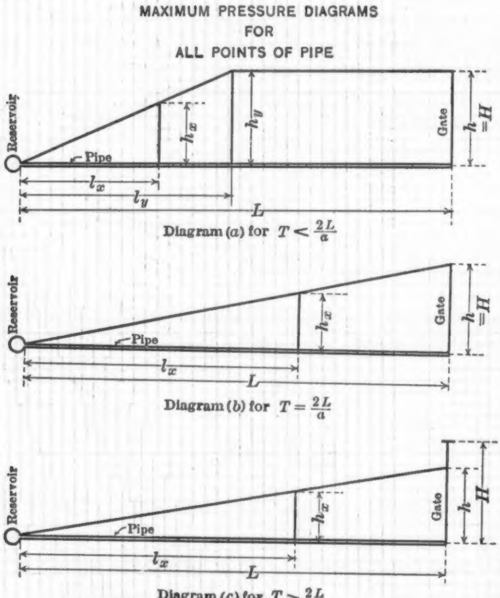

FIG. 9.

By the limitation that h can never be greater than $\frac{a V}{g}$, and, as in the last formula,

$$T < \frac{2L}{a},$$

therefore $$h > \frac{a V}{g},$$

and, by the limitation, $$H = \frac{a V}{g}.$$

For the critical point, $h = h_y,\ l = l_y,$ Mr. Vensano.

as already shown, $l_y = \dfrac{a\,T}{2}.$

Substituting,

$$h = \frac{2\,l\,V}{g\,T},$$

$$h_y = \frac{a\,V}{g}.$$

For any point between these two, the formula gives results greater than $\frac{a\,V}{g}$, and, by limitation, the absolute maximum pressure, $\frac{a\,V}{g}$, occurs at all points in this portion of the line. For all points between this critical point and the reservoir, the equation gives values decreasing uniformly to zero at the reservoir.

Case III.—(See Fig. 9 (b).)—$T = \dfrac{2\,L}{a},$

$$h = \frac{2\,l\,V}{g\,t},$$

at the gate

$$l = L,$$

$$h = H,$$

substituting the value of T, we obtain

$$h = H = \frac{\dfrac{2\,L\,V}{g}}{\dfrac{2\,L}{a}} = \frac{a\,V}{g},$$

as it should be. For all other points, values decreasing from this maximum to zero at the reservoir will be obtained.

Case IV.—(See Fig. 9 (c).)—$T > \dfrac{2\,L}{a},$

$$h = \frac{2\,l\,V}{g\,T},$$

at the gate

$$l = L,$$

and

$$h = \frac{2\,L\,V}{g\,T}.$$

For all other points, values uniformly decreasing from this maximum to zero at the reservoir will be obtained.

MINTON M. WARREN,* JUN. AM. SOC. C. E. (by letter).—The writer is greatly indebted to those who have discussed his paper. The new points of view brought out are worthy of careful study by engineers interested in water-hammer. Mr. Warren.

* Boston, Mass.

Mr. Warren.

The title of the paper and the many formulas quoted have led to some misunderstanding as to its object. The writer desired water-hammer to be the main theme, and considers his new Formula (B),

$$h = \frac{L\,V}{g\left(T - \frac{L}{a}\right)},$$ to be equal in practical importance to the common pipe formula, $b = \frac{W\,Y\,D}{2\,f_s}$ (where f_s is the allowable stress in the steel, in pounds per square foot, the other letters being as given in the nomenclature). The résumé of other ordinary water-hammer formulas was given in order to point out the limitations which they all have.

The fact that the leading authorities on hydraulics have been using water-hammer formulas which vary by 100% shows the necessity of a thorough discussion of the theoretical and experimental results from all sources, and the desirability of establishing a correct formula, based on sound theory and backed by reliable experiments.

Joukovsky was the first to do this in the case of maximum water-hammer, and, with the discussions, it is hoped that this paper may be a step in the right direction in the case of the more usual slow closing of gates, or ordinary water-hammer.

Dr. Webster's formula for surge tanks was developed with the hope that its derivation could be followed by the practical engineer. The writer has never been able to follow the mathematics used in deriving the other surge-tank formulas, and always hesitates to use a formula the derivation of which is not clear. If the derivation of a formula is unknown, the formula may contain assumptions which make it dangerous to use in certain practical cases. For example, the formulas for ordinary water-hammer are constantly being misapplied by engineers ignorant of their basic assumptions, and the results of such work are often misleading and dangerous.

Mr. Johnson's Formula (49) is simpler to solve and gives results equal to those of Dr. Webster's formula, and, to those who can follow its derivation, it is undoubtedly preferable.

At the time the paper was published, neither the writer nor Dr. Webster had seen Professor Prasil's formula for surge tanks, developed along the same lines, but with such thoroughness and wealth of calculus as to be unintelligible to the writer.

Taking up the various discussions in order, there are certain points and questions which the writer will discuss briefly.

Referring to Mr. Church's discussion (page 272): Equation (G) is referred to as "the most nearly accurate" for two reasons: First, Dr. Webster derived it with fewer assumptions than there are in the other formulas given in the paper; second, its numerical results are nearer the truth than those of any of the other formulas, with the exception of

of Mr. Johnson's Formula (49). It is derived by assuming that the friction varies directly as the velocity of the water in the pipe, which will give results too large and, therefore, on the safe side, except for possible effects of governor action. Obviously, the other formulas, which give larger results, are less accurate. (See numerical examples on page 268.) Dr. Webster's formula refers only to the special case of turning on or shutting off the full load, and does not pretend to take account of such complications as governor action. Mr. Warren.

The writer wishes to thank Mr. Church for Formula (106), which seems to give results still nearer the truth. In form, it is identical with the writer's approximate Formula (50), but the constants are different.

In regard to water-hammer, Mr. Church's Formula (113) is the ordinary one given in the paper as (47), which is accurate for use when T is considerably larger than $\frac{2\,L}{a}$. It is discussed on page 264. Referring to the discussion of water-hammer by Mr. Johnson (page 277), the writer was much interested to note that the calculations and experiments agree with his own, and that, in ordinary water-hammer, when the gate is closed slowly, the pressure rises and then continues constant, with a flat top to the pressure curve. In a recent letter to the writer, Mr. Johnson says: "I have made measurements of pressure rise due to slow closing of gates many times in many different parts of the country * * * there is always a flat top of some extent on all the pressure curves, which I have observed." This fact is a very strong argument against Joukovsky's Formula (48) which is so strongly upheld in Mr. Vensano's discussion on page 289.

As Mr. Johnson states, his Equation (6) is identical with Allievi's Equation (C), which is fully developed and discussed in the paper. For the most part, this formula checks the writer's Formula (B) very closely, but when it is used for values of T close to $\frac{2\,L}{a}$, it becomes incorrect and dangerous to use. This error is discussed on page 255 and, as may be seen from the numerical examples on page 268, the formula violates the law of conservation of energy in certain cases, giving values greater than those obtained for instantaneous closing.

Mr. Johnson restricts the use of the formula to times equal to or greater than $\frac{L\,V}{1.5\,g\,Y}$. Even this does not do away with the possibility of error, as may be seen from the following numerical cases.

Take the data on page 266, a 10-ft. pipe, $\frac{1}{2}$ in. thick, 400 ft. long, with water flowing at 6 ft. per sec.; $D = 10$, $b = 0.04$, $L = 400$, and $V = 6$.

Mr. Warren.

First, taking a static head of 5 ft., Mr. Johnson's limiting time becomes $T = \frac{L\,V}{1.5\,g\,Y} = \frac{400 \times 6}{1.5 \times 32.2 \times 5} = 10$; therefore, the formula is not applicable unless the time of closing is 10 sec. or more. In this case it would not be usable in ordinary hydraulic work where the gate closes in from 2 to 5 sec.

Now take this same case with a static head of 400 ft., and the limiting time becomes 0.125 sec. The example worked out for $T =$ 0.32 sec., on page 268, therefore, is allowable. The result is $h = 312$ ft., yet we know that as the wave has only just returned from the gate $\left(\text{as } T = \frac{2\,L}{a}\right)$, the maximum water-hammer, or $h = 465$ ft., must apply to this case. Therefore, in this case, Allievi's formula is 34% low, even with the restriction applied by Mr. Johnson.

In general, it seems to the writer that a water-hammer formula which varies its results with the static head, Y, must be inherently wrong. The gate, in closing, has merely to overcome the kinetic energy of the moving water (friction neglected) which depends on its velocity. Why a formula should give a rise of 630 ft. under 100-ft. head and, with the same data, half as much under a 400-ft. head, is hard to imagine, yet Formula (C) gives this, as is seen on page 268. Presumably, this error is the result of the inconsistency of Allievi's assumptions, pointed out on page 255.

To Mr. Trautwine the writer is indebted for a careful and complete analysis of the phenomenon of wave motion in instantaneous closing of gates. His Fig. 6, for slow closing, shows a flat top to the pressure curve the length of which depends on $\frac{2\,L}{a}$, not on T, as in the writer's formula.

In connection with Mr. Williams' discussion (page 287), the writer wishes to mention a point of special importance in water-distributing systems, which Joukovsky discovered and proved both by theory and experiment. When a small branch pipe leads off a larger pipe in which water-hammer takes place, the small pipe will be subject to double the pressure in the large pipe, provided it has a dead end. This is more or less the principle of the hydraulic ram, and is due to the fact that when the wave reaches the dead end of the small pipe, the velocity must remain constant (zero) at that point. The only means by which this is possible is by starting a reflected wave having a velocity and direction opposite to the first wave, the pressure of which is consequently the same. Adding the two pressures, we get double the pressure in the large pipe. If the small pipe has a still smaller pipe connected with it, the pressure may be quadrupled, and so on.

This phenomenon should also be carefully considered by hydraulic engineers when they take readings in instantaneous water-hammer from a pressure gauge set on the end of a small pipe leading from a penstock or wheel-case. Mr. Warren.

Joukovsky goes farther, however, and makes a statement which does not seem reasonable to the writer, namely, that when the wave returns from the dead end into the larger pipe, it may cause dangerous pressures therein. In sending the wave up the small pipe, the pressure in the large pipe is somewhat relieved, as if by a safety valve or air cushion. When the wave returns from the dead end, this fall of pressure may be more than counteracted, but it does not seem as if dangerous pressures would result, especially if there is considerable difference in the pipe sizes.

In reply to Mr. Anderson's questions on page 288, the value of K, 42 400 000 lb. per sq. ft., or 294 000 lb. per sq. in., was taken from Mead's "Water Power Engineering". It would be safer to use a minimum value for this if any data are available from which to choose it. $E = 4\,000\,000\,000$ lb. per. sq. ft., or 28 000 000 lb. per sq. in., was taken from Kent's "Handbook". A maximum value would be safer to use.

As far as water-hammer formulas are concerned, E applies only to circumferential stretching of the pipe; and expansion or other circumferential joints do not enter into it. The longitudinal joints ought not to affect it appreciably. It is worth noting that in the numerical example for maximum water-hammer, given on page 266, a variation of 10% in either E or K makes a difference of from 2 to 4.5% in the answer to the problem. In ordinary water-hammer problems, the difference would be less, and in most cases negligible.

As regards the closeness of Fig. 2 to actual pressure curves, it is true, of course, that actual graphs would contain many irregularities, but from most formulas, as well as experiments, this seems to be the general form of the curve for slow-closing gates. Minor harmonics undoubtedly occur, and may be caused by air pockets, governor action, joints, or imperfections in the pipe.

As to nomenclature, the Greek ϵ would have been preferable to e, but a has been used by Uhl and by Allievi for wave velocity.

Mr. Vensano points out on page 289, that, in Formula (H), for economical penstock diameter, B, the cost of pipe per foot of diameter per foot of length is a function of D which is unknown, and that consequently the formula should be deduced by assuming the cost of the pipe to vary directly as D^2, thus obtaining his formula.

The writer pointed out this fact on page 265, and stated certain reasons for giving the former formula the preference.

Mr. Warren.

For a first approximation, Mr. Vensano's formula might be preferable, but for final work it is not true that the penstock cost varies as D^2, owing to the commercial sizes of plates and the allowance of metal for water-hammer and rust, which are not functions of D^2.

As a matter of fact, it makes very little difference which formula is used; either one is probably as accurate as the assumptions warrant.

Referring to Mr. Vensano's excellent exposition of Joukovsky's Formula (48), the writer believes that the mathematics is correct and the theory worth the most careful consideration by engineers interested in water-hammer. The writer disagrees with the resulting formula for three reasons:

First.—One of the assumptions seems to be wrong theoretically.

Second.—Practically, the shape of the pressure curve does not agree with any experiments the writer has ever seen, and the results are much larger than those found in actual practice.

Third.—Allievi, Church, Johnson, Mead, and other authorities give formulas which yield results about half as great as those of Joukovsky.

The seeming error in assumption is as follows: Mr. Vensano assumes that when the wave arrives at the gate, it is perfectly reflected, and the pressure falls, not only to zero, but below it, just as a pendulum swings across zero and up an equal distance on the other side. The first wave is started by the gate closing a small increment. This wave, though started by a local disturbance, spreads across the pipe, returns from the reservoir, and finds the gate still partly open. Therefore, only the part of the wave that encounters the gate reflects from it, the rest passes on into the turbine or open air, and its reflection is seriously impaired.

The perfect reflection encountered at the reservoir is due to the fact that the pressure there must be constant (atmospheric), though the velocity may vary. Similarly, at a dead end or closed gate the total velocity must be constant (zero), though the pressure may change.

At a partly open gate, however, as the pressure rises (due to the wave), the water shoots out at a higher velocity and relieves it somewhat. Certainly, the velocity is not constant or independent of the returning wave at the part of the gate that is still open, and consequently perfect reflection cannot take place.

As the gate approaches nearer and nearer to its closed condition, this argument has less and less weight, but there is another point worthy of attention. All experiments tend to show that, although the positive waves in pipes act very much as they should according to theory, the negative waves are not equally accommodating and, in fact, rarely fall much below zero, however great the previous rise may have been. This may be seen very clearly by a glance at the actual graphs obtained in Joukovsky's experiments and reproduced in Simin's article.

Mr. Warren.

The writer feels, however, that some slight reflex wave action may be present, and may account for some of the experimental numerical results which are greater than would be indicated by Formulas (B) or (47), as well as for some of the irregularities and harmonics noted in experiments. In other words, these waves, although not nearly as important as Joukovsky's formula would indicate, may explain discrepancies between Formula (B) and actual experiments.

If Mr. Vensano's deductions could be backed by actual experimental graphs, he would have a strong case, but as the writer has never before heard of a case where, in slow gate closing, the pressure fluctuated violently from zero to h, and as Mr. Johnson, in many practical cases, has always observed a flat top to the pressure curve, it would seem that the theory on which Mr. Vensano's formula is based, must be incorrect.

In conclusion, the writer wishes to emphasize again the need of a general formula for water-hammer which will be applicable to the whole range of the phenomenon and which is, theoretically as well as practically, correct. Formula (B), if backed by careful experiments, would fill this need, and the writer hopes that some engineer who has the opportunity will make such experiments and present the results to the Society.

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TRANSACTIONS

Paper No. 1338

THE WATER-PROOFING OF SOLID STEEL-FLOOR RAILROAD BRIDGES*

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WITH DISCUSSION BY MESSRS. ALBERT J. HIMES, WILLIAM S. BABCOCK, JONATHAN JONES, J. LEE ALLEN, GLENN B. WOODRUFF, G. J. RAY, J. B. W. GARDINER, HENRY H. QUIMBY, RALPH J. LAWRENCE, A. T. GOLDBECK, F. AURYANSEN, J. H. FICHTHORN, EDWARD W. DE KNIGHT, ALBERT H. RHETT, A. W. CARPENTER, C. T. DELAMERE, JOHN JERVIS VAIL, PHILIP B. WALKER, CHARLES RUFUS HARTE, AND SAMUEL TOBIAS WAGNER.

SYNOPSIS.

The object of this paper is to submit a specification for the design, materials, and manner of application of the water-proofing of railroad bridges with solid steel floors.

Present practice in bridge construction of this type differs so much among engineers, as to design and materials used, that it is hoped that the discussion from those who have had experience with work of this class will be of value in determining on more uniform practice in a detail of modern bridge construction which is yearly commanding more attention.

The specifications, as presented, have been the outgrowth of about eight years' experience with structures of this class, and with a number of kinds of materials and design. Up to the present time, the speci-

* Presented at the meeting of January 6th, 1915.

fications which have been used have been tentative, and based on the best information which could be obtained. It seems, however, that there are certain principles—more or less fixed—which may be lost sight of by the inexperienced designer, and the chief object in preparing this paper is to state specifically what they are believed to be, with the hope that the discussion will either refute or approve them.

The specifications are not intended, in any way, to cover the design of the structural work, but to call attention to those details which should be borne in mind by the designer when engaged on a structure, of the type under consideration, which has to be water-proofed. It is believed to be of great importance that the structural designer shall have in mind the fact that the water-proofing plays an important part in the details, and that it should be co-ordinated with the design as he proceeds.

Six Appendices are attached to this paper, for the purpose of adding such information as has been obtained in connection with the details of the subject. Some of the data they contain have not been published heretofore; they have been obtained in an effort to arrive at the truth on lines in which there seemed to be little precedent. Appendix F shows the practice which has been followed in the past six or seven years on work with which the writer has been connected.

The matter thus shown is, briefly, as follows:

Appendix A—Tests of Asphalts;
" B—Examination of Felts and Fabrics;
" C—Examination of Rock Asphalts;
" D—Data as to Use of Asphalts in the United States in 1912;
" E—Abstracts from Paper, by Mr. J. W. Howard, on Asphalts;
" F—Various Plans Used in Actual Practice.

Some explanation is first due as to the reasons for introducing certain features. The water-proofing of solid-floor railroad bridges is undertaken for two reasons:

First.—To protect the metal of the floor from corrosion due to the alternately wet and dry condition of a ballasted floor;

Second.—To prevent the water, which is absorbed by the ballast and given off slowly, from dripping on a street beneath.

A reasonable depth of girders and floor makes a stiff structure, and aids in making it water-tight.

There can be but little doubt that, in all designs, the primary object should be to provide such details as will remove the water from the bridge in the quickest possible time and by the shortest route; and further, in case of the possibility of the water being held on the bridge for any considerable time after a storm, to design the height of the water-proofing so that it will be above the highest water level. There are many places where the ballast will become very dirty in time, and at such localities this detail is important.

It is also considered inadvisable to throw all the water off the bridge, at its ends, over the back walls. It is very difficult to design at this point a satisfactory detail which will prevent water from forcing its way back over the back wall and down the face of the abutment, even though the drainage back of the abutment is specially good. It is much better to collect the water on the deck of the bridge, take it below the floor by inlets and down-spouts, and discharge it below the floor on the ground, or into a sewer (if in a city). Such inlets can be made so as to be accessible from below for cleaning, and, with ordinary care, can be made satisfactory as far as leakage around them is concerned. To carry the water away quickly, there should be a number of these inlets on any bridge of considerable extent.

In the case of deck structures, the water-proofing details are simple, the principal questions to be settled being the type of water-proofing to be used and the durability of the materials. The application of the water-proofing—whatever kind may be selected—is not difficult.

In a half-through structure the conditions for water-proofing are at their worst. If the top of rail is near the top of girder, it is generally advisable to carry the water-proofing over the girder and encase its entire top in concrete. The writer wishes that a number of such girders, in which he has assisted in the design and construction, had been treated in this manner.

The finish of the water-proofing against the web of a half-through girder, the top of which is 3 ft. or more above the top of rail, is one of the most difficult problems to be solved. In some cases, it is the practice to carry the water-proofing nearly if not all the way up to the top of the girder, and protect it with a considerable thickness of concrete, but, generally, it is finished up against the web not far above the

level of the top of rail. This finish, if done satisfactorily, will make a good piece of work of the structure as a whole, provided the detail in respect to water running over the back walls is guarded against. It is believed that riveting a flashing angle to the web does not provide a satisfactory detail, as it is impossible to make it continuous on account of stiffeners and gussets. Placing a pocket of ductile, adhesive asphalt (which will not harden in cold weather) as a joint between the water-proofing layer and the web is believed to give the best results, and in case of the deterioration of this asphalt, the detail is in a place where repairs can be made without interfering with traffic. The question of interference with traffic in making water-proofing repairs is one that is giving many railroad bridge engineers serious concern, and is leading slowly, but surely, to much more elaborate, effective, and expensive systems, in order that repairs may be postponed as long as possible. It is possible to make the proper kind of pure asphalt, unmixed with foreign matter, adhere to a clean surface of metal or concrete, and thus secure a tight joint. It has been found advisable to protect this pocket of asphalt with a layer of concrete to prevent the ballast, which may be carelessly thrown against the girder, from cutting into it. (See Figs. 8 and 9, Appendix F.)

As to materials to be used for the solid bridge floor, it is believed that, after the question of durability is disposed of—as one of the most important items—the most durable material, which can be shown to possess adhesion and ductility at low temperatures, and at the same time be sufficiently hard to prevent running at maximum exposure temperatures, is the most desirable—say, temperatures generally between 0° and 200° Fahr. Higher melting points are undesirable, for a number of reasons, and should not be used. The most critical time to test a bridge floor is during melting snow, and cracks which may develop at that time are sure to be channels for water.

Whether felt, fabric, or asphalt mastic shall be used, is a debatable question, and will depend somewhat on conditions. If hard pressed for head room in depth of floor construction, the use of asphalt mastic is indicated, and, though it is generally considered an undesirable material, it is believed that, with proper natural rock, proper flux, and careful mixing and placing, first-class results can be obtained, if the pure asphalt seal is used against the metal. A large bridge, of about 43 000 sq. ft., lately built, is water-tight, and recently the writer has

seen materials of this character in splendid condition after 20 years' service.

Felts or fabrics, of course, contain organic matter, and, if not entirely covered with asphalt, are subject to disintegration by "rotting". It would seem, however, that, if properly prepared and laid, these materials should form a very satisfactory membrane—more satisfactory, as far as elasticity is concerned, than the mastic. A felt of inorganic material—asbestos—has all the proper qualities, from the standpoint of durability. It is generally laid in combination with a layer of treated burlap to give it additional strength. Practice sometimes protects asbestos felt with asphalt mastic instead of brick or reinforced concrete. This is probably allowable with asbestos felt, but should never be used with any of the other felts. Experience shows that hot mastic, placed on a layer of felt, or fabric treated with asphalt, will draw the asphalt out of the felt and incorporate it as part of the mastic.

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GENERAL SPECIFICATIONS FOR WATER-PROOFING SOLID-FLOOR RAILROAD BRIDGES.

PART I. DETAILS OF CONSTRUCTION.

1.—Depth.—The depth of steel or concrete construction shall be such as to allow a sufficient distance from top of rail to top of steel or concrete floor for proper water-proofing, and protection from the cutting action of the ballast. Under ordinary conditions, a depth of from 3.5 to 4.0 ft. from top of rail to clearance line below is sufficient.

2.—Drainage Grade.—Provision shall be made for grades of at least 1% on the floor of the bridge, to remove water promptly. Where this cannot be done in the steelwork, cement concrete, with a minimum thickness of 2½ in., shall be placed so as to drain the water to the inlets.

3.—Inlets.—Cast-iron inlets shall be set at proper places in the floor and provided with movable top grates. The down-spout from each inlet shall be provided with a trap and clean-out, which shall be accessible from below the bridge. The down-spout shall be of wrought iron, and connected to a sewer or arranged according to local conditions.

4.—Details of Steelwork.—Where two longitudinal girders meet over a column, the end stiffeners shall be placed with the outstanding leg toward the center of the girder, and the girders shall be connected with ⅜-in. plates on each side of the web. A plate shall also be placed covering the joint on the top flange.

5.—Stiffening angles on the outstanding edges of gusset-plates in half-through or through girders shall be terminated below the level of the water-proofing, the gusset-plates shall be heavier, and shall be placed from 10 to 12 ft. apart.

6.—Where the top of girder approximates the same height as top of rail, the water-proofing and protection shall cover the entire top of the girder.

7.—The apron-plate from the steel floor over the back wall shall be provided with a curb angle against which to finish the water-proofing, and to this angle shall be riveted a vertical plate to prevent dirt from collecting under the apron-plate. The apron-plate shall be anchored to the back wall.

8.—Water-proofing.—General Design.—On top of the prepared surface of the concrete shall be placed either of the following:

1st.—One or more thicknesses of felt or fabric, of quality and applied as specified hereafter, together with proper protection;

2d.—Asphalt mastic at least 1½ in. in thickness, of quality and applied as specified hereafter.

9.—Felt, Burlap, or Fabric.—When water-proofing of this kind is to be used, either of the following types shall be adopted:

1st.—From four to six layers of felt;
2d.—One middle layer of treated burlap, with four layers of felt;
3d.—One layer of felt, two layers of burlap, and two layers of felt;
4th.—One middle layer of treated burlap, and two layers of asbestos felt;
5th.—Either one or two layers of treated cotton-drill fabric.

10.—Protection of Water-proofing.—After the completion of the felt or fabric water-proofing, the entire surface shall be covered and protected by one of the following methods:

1st.—Straight, hard-burned brick laid flat, with joints filled either with water-proofing compound or cement grout. Water-proofing compound should only be used as a filler on flat or nearly flat surfaces;
2d.—A layer of cement concrete from 2 to 2½ in. thick, with wire reinforcement;
3d.—A layer of about 1½ in. of asphalt mastic used only on top of asbestos felt. It is believed that no protection is necessary for the asphalt mastic.

11.—Special Drainage over Protection.—On top of the protection coat, and outside the line of the ties, a line of half-round cast-iron pipe, 6 in. in diameter, and perforated frequently, shall be placed to collect the water and convey it to the inlets.

Part II. Preparation for Water-Proofing.

12.—Preparation of the Steel.—All openings in the steelwork shall be thoroughly closed, either by caulking with burlap dipped in hot asphalt, or by the use of sheet metal sufficient to maintain the concrete base before applying the burlap and asphalt.

13.—Preparation for Water-proofing Materials.—Wherever called for by the plans, the decks of the bridges shall be protected with 1:3:5 concrete, with ¾-in. stone or gravel, mixed as specified hereafter, finished with a 1:2 mix of cement mortar, ½ in. thick, troweled to a smooth surface on top, as shown. This concrete shall be allowed to dry thoroughly so as to prevent the formation of steam when the hot water-proofing materials are applied.

14.—All vertical or sloping surfaces of concrete or steel shall be thoroughly cleaned of dust, dirt, loose particles, paint, and grease. The use of a hand-bellows is recommended for cleaning loose dust and dirt from the surfaces. For cleaning paint and grease from the steel, and freshening the surfaces of asphalt, where a junction of old and new is to be made, or where a pocket of pure asphalt is used against

the girders and the felt or mastic, gasoline shall be used, either by swabbing the surface with it, or by pouring a small quantity over the surface to be cleaned and setting fire to it. The use of a blow-lamp is also recommended.

15.—These surfaces shall then be painted with two coats of approved asphalt, diluted with gasoline. The materials of the first coat shall be proportioned so as to give a brownish tint. The second coat shall have a larger quantity of asphalt.

16.—Both coats of paint shall be thoroughly applied and worked into the surfaces, so as to give a uniform coating of the asphalt.

17.—Paint shall not be applied to damp concrete or steel. The painting shall be done immediately in advance of the application of the water-proofing materials and before dust has had time to collect.

18.—If the concrete is damp before the water-proofing is applied, the surface shall be first covered with a 2-in. layer of hot sand and allowed to stand for from 1 to 2 hours, after which the sand shall be swept back, uncovering sufficient surface to begin work, and the operation repeated over a new surface.

19.—Concrete Proportions.—The cement concrete shall be proportioned by measurement of volumes. The volume of a barrel of cement, 376 lb., shall be assumed to be 3.6 cu. ft. The sand and stone shall not be packed more closely than by throwing, in the usual way, into a barrel or box at the time of measurement.

20.—Cement.—Portland cement shall be used, of the quality specified by the American Society for Testing Materials.

21.—Stone.—The stone shall be clean, hard, crushed stone, or pebbles, to be approved by the Chief Engineer, and shall be composed of the whole run of the crusher, from ¼ in. to ¾ in. in size, screened of dust and particles less than ¼ in. in greatest dimension.

22.—Sand.—The sand shall be clean and sharp, and shall be composed of grains graded from "fine to coarse," screened to reject all particles of a greater diameter than ¼ in. It shall be free from foreign matter, and subject to the approval of the Chief Engineer.

23.—Care of Sand and Stone.—Sand and stone, when delivered on the work, shall be dumped on platforms, and not on the ground.

24.—Hand Mixing.—When mixed by hand, the cement and sand shall be first mixed dry and made into a mortar. The stone shall be spread on a suitable floor to a depth of about 6 in., thoroughly wetted, and the mortar evenly spread over it, care being taken that the stone of each batch is mixed as to size. The whole mass shall then be turned over four times and raked, to secure complete and uniform mixture. If the Contractor desires to use some other method, he shall submit it for approval. Should the mixture be permitted to set before placing or tamping, it shall be removed and not used. Hand-mixed batches shall not be larger than 1 cu. yd. in volume.

25.—Depositing.—All concrete shall be deposited as the Chief Engineer shall direct. It shall be of such consistency that when dumped in place it shall not require much tamping, and shall be laid with a view to be an aid to the water-tightness of the structure, and not merely a support for the water-proofing materials. All showing surfaces shall be troweled to a smooth, hard surface.

26.—In cases where concrete haunching against girders is called for by the plans, forms shall be used, and the concrete shall be of a wet consistency.

Part III. Materials and Application.

Water-Proofing Felt or Fabric and Asphalt.

27.—Materials.—On the prepared surface, apply the specified number of layers of approved saturated and coated felt, with a finished surface, and weighing about 14 lb. per 100 sq. ft.

28.—The bids shall be based on the use of the type of felt specified in Paragraph 27, but additional alternate bids will be considered, based on felts or fabrics other than these, which may be approved by the Chief Engineer. In the event of such alternate bids being made, the bidders shall present with them sufficient data as to the methods of manufacture, quality of materials, and references to places where such felts or fabrics have been used, giving dates of application.

29.—All materials shall be delivered on the work in their original packages, and properly branded.

30.—The acceptance or rejection of an asphalt shall rest with the Chief Engineer, and shall be based on the requirements stated in Paragraphs 31 to 37.

31.—The asphalt used shall consist of fluxed natural asphalt, or asphalt prepared by the careful distillation of asphaltic petroleum.

32.—It shall contain, in its refined state, not less than 98% of bitumen soluble in cold carbon-disulphide. The remaining ingredients shall be such as not to exert an injurious effect on the work.

33.—When 20 grammes are heated for 5 hours at a temperature of 325° Fahr., in a tin box $2\frac{1}{2}$ in. in diameter, it shall not lose more than 2% by weight, nor shall the penetration at 77° Fahr., after such heating, be less than one-half of the original penetration.

34.—The melting point shall be between 150° and 190° Fahr.

35.—A briquette of the solid bitumen, having a cross-section of 1 sq. cm., shall show ductility at 40° Fahr., and at a temperature of 77° Fahr. shall show ductility of not less than 20 cm., the material being elongated at the rate of 5 cm. per min. (Dow moulds.)

36.—All tests shall be conducted according to methods approved by the Chief Engineer.

37.—The penetration indicated herein refers to the depth of penetration, in hundredths of a centimeter, of a No. 2 cambric needle, weighted to 100 grammes, at 77° Fahr., acting for 5 sec.

38.—*Application.*—All flashing and reinforcing around inlets and other places specified shall be carefully executed.

39.—Water-proofing shall not be done in wet weather, or at a temperature below 32° Fahr., without special orders from the Chief Engineer. The felt shall be laid shingle fashion, the first two layers longitudinally and the last three transversely to the center line of the bridge, where five layers are called for, and as specified in detail in other cases, and shall be carried up the haunching and made secure against the girder in a satisfactory manner, or as shown on the plans. The flashing against vertical or inclined surfaces shall be in accordance with the directions of the Chief Engineer, if not indicated. The first layer of felt shall not be cemented to the floor of a steel bridge, except around the drain outlets. On an arch bridge, the first layer shall be cemented to the top of the arch. At no point shall there be less than the specified number of thicknesses.

40.—As the hot asphalt is spread, the felt shall be immediately rolled into it, and rubbed and pressed over its surface, so as to eliminate air bubbles and insure thorough sticking. One mopful of the asphalt shall not be spread over more than 1 sq. yd. of surface at one mopping. Not less than 2.5 to 3 gal. of asphalt shall be used on 100 sq. ft. of a single layer of felt. The top layer shall also be mopped and the work done so that the layers shall be one compact mass.

41.—The finish of the water-proofing against the girders or concrete shall be made with a pocket of pure elastic asphalt of the quality specified in Paragraphs 31 to 37, except that the melting point shall be between 140° and 180° Fahr., the ductility at 40° Fahr. shall be at least 3 cm., and the adhesive qualities shall be satisfactory to the Chief Engineer. The surfaces with which this material comes in contact shall be dry, absolutely free from dust or grease, and, previous to its application, shall be covered with a thin paint made by dissolving the asphalt in gasoline, as specified in detail in Paragraph 15.

42.—Particular care shall be taken to make a tight joint around gussets, stiffeners, and the ends of girders.

43.—Care shall be taken to prevent injury in any way to the water-proofing by the passing of men or wheel-barrows over it, or by throwing any foreign materials on it.

44.—After the water-proofing course has been completed, the horizontal surfaces shall be protected, as shown on the plans, by a course of straight, hard-burned and dense brick, laid flat in a bed of 1 to 3 cement mortar, with full joints. There shall be not less than ½ in. of mortar between the felt and the bricks. The brick shall not increase in weight more than 10% when immersed in water for 7 hours.

45.—The haunching, and about 18 in. in width of the horizontal surface adjacent to the haunching, shall be protected, as shown on the plans, by about $2\frac{1}{2}$ in. of 1 : 3 : 5 concrete, reinforced with No. 8 and No. 10 wire cloth, electrically welded, having a 3 by 8-in. mesh.

46.—Every care shall be taken to insure satisfactory and thoroughly water-tight joints between the main layer of water-proofing and the girders; and special attention shall be given to stiffeners, gussets, etc. The water-proofing shall also be carried down over the back walls to below the elevation of the bridge seat, as shown on the plans, or as directed.

47.—Rolls of felt shall be stored on end, and not laid on their sides.

48.—Water-proofing shall be done only by experienced and expert felt water-proofers.

Natural Rock Asphalt Mastic.

49.—*Rock Asphalt Mastic.*—Wherever called for by the plans, the decks of bridges shall be water-proofed with natural rock asphalt mastic, as specified in Paragraph 50.

50.—The cement concrete, prepared as specified heretofore, shall be water-proofed with asphalt mastic equal in quality for the intended purpose, as to ingredients used and resistance to water, to the following specifications, and be approved as such:

Sicilian rock asphalt mastic	60 parts.
Clean, sharp, graded grit and sand to pass a sieve of 8 meshes per inch	30 parts.
Asphalt as specified in Paragraphs 30 to 37	10 parts.

These proportions shall be varied when required by special conditions on the work.

51.—The mixture shall be made at the site of the work, shall be heated to a temperature of from 250 to 300° Fahr., and shall be stirred until all the ingredients are thoroughly incorporated. It shall then be spread and thoroughly worked, to free it from voids, and shall be ironed to a smooth surface with smoothing irons, if so directed. All mastic shall be applied in two coats, making the total thickness shown on the plans. The two coats shall break joints, and the mastic shall be distributed evenly. Where the thickness of the concrete plus mastic is less than $2\frac{1}{2}$ in., the full thickness shall be made up of asphalt mastic.

52.—All mastic delivered on the work shall be properly branded. Water-proofing shall not be done in wet weather, or at a temperature below 32° Fahr., without special orders from the Chief Engineer.

53.—Pockets of asphalt shall be placed against all metal, and mastic along girders, around stiffeners, gussets, etc., as specified in detail in Paragraphs 14 to 18, inclusive, and 41.

54.—Great care shall be taken around expansion joints, drain-pipes, and similar places, where a separation may take place.

55.—After the mastic is laid, it shall be mopped with pure melted asphalt, and the surface shall be spread with a layer of clean, coarse sand, to harden the top.

56.—The pockets of asphalt placed against the girders, stiffeners, and gussets shall be protected, as shown on the plans, by about $2\frac{1}{2}$ in. of 1:3:5 concrete, reinforced with No. 8 and No. 10 wire cloth, electrically welded, having a 3 by 8-in. mesh.

General Conditions.

57.—General Conditions.—The furnishing and erection of the steelwork for the bridge to be water-proofed will be executed under a separate contract, and the riveting will be completed, the erection finished, and the steel floor cleaned up, ready for the water-proofing, before the work on this contract is begun. In addition to the foregoing, the Contractor shall make a final cleaning of the steelwork before the work of water-proofing is begun.

APPENDIX A.

Tests of Asphalts. (See Table 1.)

The following notes are to be considered in connection with the interpretation of the tests recorded in Table 1.

Bitumen Soluble in CS_2.—This represents the total bitumen soluble in CS_2, and is determined by the method recommended by the American Society for Testing Materials. Generally, it may be said that 99.5% of the petroleum asphalts are soluble in CS_2. Others vary, especially the natural asphalts, some of which may run as low as about 55%, although gilsonite will dissolve to the extent of more than 99 per cent.

Bitumen Soluble in 88° P.E.—Hubbard* says:

"The term asphaltenes is commonly applied to bitumen insoluble in petroleum naptha [or ether] and malthenes to that portion which is soluble. It is of course evident that both of these terms cover a multitude of compounds, but in general it may be said that the asphaltenes tend to give body and consistency as well as adhesive properties to the products in which they are found, so that this determination serves as an indication of the mechanical stability of the material as well as its binding qualities."

The figures given are the percentages of the total bitumen thus soluble in 88° P.E.

Ductility.—This test is made by filling with asphalt a mould which has a cross-section of 1 sq. cm., soaking it in water at the desired temperature for 20 min., and then pulling it apart at the rate of 20 cm. per min. The elongation at the moment of rupture is called the ductility, and is expressed in centimeters.†

The ductility at different temperatures indicates the ability of the material to remain firm and elastic under extremes of weather. The less the range of ductility at several temperatures, the better the material for most purposes.

Penetration.—Penetration tests are made by measuring the penetration of a No. 2 standardized weighted needle at different temperatures, and under definite weights and times.

In such tests, the following weights and times are used:

At	40°	Fahr.		200	grammes	acting	for	60 sec.
"	77°	"		100	"	"	"	5 "
‡"	77°	"	(after heating)	100	"	"	"	5 "
"	110°	"		50	"	"	"	5 "

The penetration is expressed in hundredths of a centimeter.

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* "Dust Preventives and Road Binders," p. 361.

† The details of the method are given in *Proceedings*, Am. Soc. C. E., for December, 1914, p. 3047.

‡ This test is made after heating for 5 hours to a temperature of 325° Fahr.

TABLE 1.—TESTS OF WATER-PROOFING ASPHALTS.

Test No.	Bitumen Soluble in:		Ductility, in Centimeters, at:			Penetration at:			Loss on heating for 5 hours at 325° Fahr.	Flow point, in degrees Fahr.	Melting point, in degrees Fahr.	Remarks.
	CS_2	88° P.E.	40° Fahr.	77° Fahr.	110° Fahr.	40° Fahr.	77° Fahr.	After 5 hours heating at 325° Fahr.				
1-*a*	99.48	......	2.0	3.0	3.5	40	36	26	0.12%	......	......	
1-*b*	99.88	......	2.5	3.0	......	18	40	....	0.38%	......	266	
1-*c*	99.91	67.40	......	2.5	......	28	29	28	0.31%	230	270	
1-*d*	99.83	63.11	1.0	3.5	4.0	29	28	21	0.45%	244	275	
1-*e*	99.73	60.43	0.75	3.5	4.5	15	20	15	0.65%	194	228	
1-*f*	99.92	61.89	0.75	3.75	5.25	16	23	22	0.83%	154	228	
1-*g*	99.74	66.04	1.00	2.00	3.5	33	41	32	1.90%	176	212	
2-*a*	98.33	......	1.00	too	soft	54	163	70	0.27%	......	......	
2-*b*	99.88	54.37	4.6	over	100	18	over 180	180	0.11%	72	91	
3-*a*	97.41	......	1.5	2.0	3.5	37	33	24	0.72%	......	......	
3-*b*	95.89	24.62	0.0	4.2	too soft	35	86	33	2.99%	......	119	
3-*c*	99.04	36.12	3.0	3.5	1.5	38	46	36	1.11%	194	234	
3-*d*	98.93	36.28	2.5	3.0	1.5	40	43	37	0.72%	216	250	
4-*a*	99.37	......	3.0	8.0	......	13	49	35	0.30%	......	180	
5-*a*	99.41	......	0.5	2.5	3.0	18	24	14	0.25%	......	250	
5-*b*	98.97	......	3.5	8.0	17.0	42	57	40	0.68%	......	169	
5-*c*	98.95	......	4.0	13.0	......	41	49	49	0.14%	......	168	
5-*d*	99.38	......	4.0	17.0	......	19	48	36	0.17%	......	185	
5-*e*	99.80	......	......	9.0	......	25	60	38	0.23%	......	176	
5-*f*	99.60	68.97	4.0	7.0	18.0	31	49	32	0.26%	138	155	
5-*g*	99.33	60.09	0.0	2.5	3.5	12	15	15	0.03%	212	262	
5-*h*	99.84	68.72	0.0	8.5	27.5	37	44	32	0.19%	158	187	
5-*i*	99.56	69.94	0.0	15.5	30.0	31	47	35	0.36%	131	169	
5-*j*	99.66	70.42	3.0	9.0	42.0	35	40	30	0.12%	113	163	
5-*k*	99.64	70.77	3.5	17.0	100.0	28	49	34	0.50%	149	176	
5-*l*	99.71	67.93	3.5	12.0	53.0	31	42	30	0.36%	147	174	
5-*m*	100.00	85.45	4.0	12.0	50.0	14	35	23	......	140	153	
5-*n*	99.66	67.20	2.0	2.5	3.0	40	43	38	1.33%	217	248	Identity of sample questioned.
5-*o*	99.96	65.20	4.0	8.5	16.0	24	28	25	0.67%	131	225	
5-*p*	99.78	74.14	4.5	29.5	68.0	39	58	31	0.63%	117	146	
6-*a*	99.59	69.97	4.5	10.5	26.0	32	42	26	0.37%	149	176	
6-*b*	99.75	56.78	4.0	19.0	51.0	46	51	29	0.10%	131	163	
6-*c*	99.78	71.70	7.0	53.0	100.0	32	52	32	0.72%	131	158	
6-*d*	99.92	85.45	4.5	12.0	50.0	14	35	23	......	140	153	

TABLE 1.—(*Continued.*)

Test No.	Bitumen Soluble in:		Ductility, in Centimeters, at:			Penetration at:			Loss on heating for 5 hours at 325° Fahr.	Flow point, in degrees Fahr.	Melting point, in degrees Fahr.	Remarks.
	CS_2	88° P.E.	40° Fahr.	77° Fahr.	110° Fahr.	40° Fahr.	70° Fahr.	After 5 hours heating at 325° Fahr.				
7-*a*	99.19	......	0.0	3.0	14.0	11	20	14	0.48%	......	203	
7-*b*	99.92	70.68	2.5	5.0	......	32	45	....	......	......	207	
7-*c*	99.86	71.70	3.0	5.5	......	33	48	....	......	......	201	
7-*d*	99.67	72.11	3.0	5.7	10.5	39	51	40	0.32%	163	192	
7-*e*	99.74	69.51	2.5	5.0	13.5	28	40	40	0.51%	167	198	
7-*f*	99.72	45.94	3.0	6.0	20.0	55	53	34	0.35%	149	190	
7-*g*	99.31	67.32	2.5	6.5	11.0	17	30	25	0.48%	140	183	
8-*a*	99.78	80.27	0.0	45.0	70.0	28	47	21	0.54%	115	150	
8-*b*	99.71	65.34	2.5	7.5	13.0	20	34	23	0.66%	176	205	
8-*c*	99.81	69.95	2.5	5.5	10.0	46	46	30	0.40%	158	201	
8-*d*	99.22	67.42	4.5	8.5	17.2	21	40	30	0.54%	126	171	
9-*a*	98.85	......	2.0	3.0	4.0	46	57	42	0.47%	......	228	
9-*b*	98.40	64.64	0.0	1.5	2.0	13	16	15	0.07%	212	260	Cracked after heating 5 hours.
9-*c*	98.72	75.49	2.0	3.0	4.0	51	69	55	0.35%	149	200	
10-*a*	98.98	92.38	2.5	6.0	15.5	31	37	36	0.22%	......	160	
11-*a*	98.09	64.72	0.0	2.5	4.0	17	20	15	0.35%	......	244	
12-*a*	99.56	75.14	0.0	10.0	24.0	28	50	29	0.33%	140	156	
13-*a*	96.74	......	2.3	3.0	......	38	42	....	......	......	252	
13-*b*	99.74	52.25	4.0	14.0	30.0	50	64	38	0.44%	133	163	
13-*c*	99.71	67.38	5.0	18.5	48.0	50	64	32	0.63%	122	149	
13-*d*	99.51	74.62	5.5	8.5	22.0	33	60	47	0.09%	129	158	
13-*e*	98.62	80.76	5.00	9.00	38.0	34	69	56	0.20%	117	163	
14-*a*	99.82	67.40	0.5	1.5	3.7	32	27	25	0.43%	212	266	
14-*b*	99.57	66.30	0.5	1.0	1.8	36	26	20	0.47%	250	297	
14-*c*	99.64	75.43	3.5	7.5	11.2	50	64	54	0.27%	167	196	
14-*d*	95.88	75.76	4.00	6.5	11.0	31	57	49	0.25%	122	172	
15-*a*	99.80	70.79	2.5	3.5	5.0	43	53	43	0.44%	208	230	
16-*a*	61.76	59.21	0.0	0.5	8.0	0.5	4	1	1.22%	200	225	Refined Trinidad.
17-*a*	93.00	75.33	0.0	24.5	37.5	3	20	8	2.68%	130	145	Refined Bermudez.
18-*a*	99.90	67.57	0.5	2.3	3.5	26	54	42	0.70%	167	199	

Loss on Heating.—This is a volatilization test, and is made by heating a sample in the air oven for 5 hours at a temperature of 325° Fahr., and then expressing the loss as a percentage. It is believed, by investigation of the subject, that the loss of weight thus ascertained is a fair comparative indication of the loss by volatilization suffered in the course of time on exposure. It is made in a similar manner to that described by Richardson.*

Melting Point.—This is determined by preparing ½-in. cubes of the material, and measuring the temperature.†

The melting point of an asphalt is very closely related to its hardness or brittleness, but varies considerably. The determination of the proper melting point depends, almost entirely, on the conditions under which the material is to be used and how it is to be mixed and applied. High melting point materials cannot be used to apply in thin layers to cold materials, on account of the rapidity with which they cool, but can be used in mastic mixtures, if the other ingredients are hot. It is generally true that materials with a high melting point will be deficient in ductility at low temperatures.

In the tables of tests, the numbers, 1, 2, etc., represent different brands of prepared asphalt; the letters, *a*, *b*, etc., refer to different samples.

* "The Modern Asphalt Pavement."

† By the method described in the Report of the Special Committee on Materials for Road Construction, *Proceedings*, Am. Soc. C. E., for December, 1914, pp. 3043 and 3044.

APPENDIX B.

Felts and Fabrics. (See Table 2.)

For use in solid bridge floor water-proofing, there are a large number of felts, burlaps, and fabrics, which are treated in the shops with various asphaltic compounds, and are used to make up a compound membrane in the field by fastening the several layers together with some asphalt compound applied hot.

The tests in Table 2 are representative of such materials, as prepared by nine different manufacturers. They are intended to show the percentages of bitumen and felt or fabric per square foot, as well as the total bitumen, and its solubility in 88° P.E.

TABLE 2.—Tests of Water-Proofing Felts and Fabrics.

No.	Description.	Percentage of Bitumen.		Original weight, in ounces per square foot.	Bitumen, per Square Foot.		Felt, Etc., per Square Foot.		Tensile strength, in pounds per inch of width.
		Total.	Percentage soluble in P.E.		Ounces.	Percentage.	Ounces.	Percentage.	
1-*a*	Saturated and coated burlap..........	71.47	63.78	2.88	2.06	71.47	0.82	28.53	60
1-*b*	Saturated felt..........	57.76	73.55	2.31	1.33	57.76	0.98	42.24	20
1-*c*	Saturated and coated felt..........	72.81	55.39	2.32	1.69	72.81	0.61	26.51	25
2-*a*	" " " "	61.13	73.06	2.29	1.40	61.13	0.88	38.39	30
2-*b*	" " " burlap..........	61.15	52.65	2.19	1.45	66.20	0.74	33.80	58
2-*c*	" " " felt..........	56.35	47.25	2.66	1.66	62.40	1.00	37.60	18
2-*d*	" " " "	57.90	73.40	2.74	1.59	58.00	1.15	42.00	
3-*a*	" " " "	66.69	69.92	2.24	1.49	66.69	0.74	33.14	35
3-*b*	" " " "	74.05	58.60	2.54	1.98	77.90	0.56	22.10	20
4-*a*	" " " "	58.88	69.53	2.75	1.62	58.88	1.03	35.52	60
4-*b*	" " " burlap..........	58.80	......	2.43	1.43	58.75	1.00	41.25	
4-*c*	" " " felt..........	52.48	......	1.90	0.99	52.50	0.90	47.50	
5-*a*	Saturated asbestos felt..........	......	......	2.84	1.72	60.50	1.12	39.50	
5-*b*	Saturated burlap..........	......	......	1.04	0.69	66.90	0.35	33.10	
5-*c*	" "	52.78	77.85	1.49	0.79	53.02	0.70	46.98	
5-*d*	Saturated asbestos felt..........	27.17	76.81	1.86	0.51	27.42	1.35	72.58	
6-*a*	Saturated felt..........	53.2	78.25	2.26	1.20	53.20	1.06	46.80	44
7-*a*	" "	59.35	......	2.61	1.55	59.40	1.06	40.60	
8-*a*	" "	59.20	73.83	2.72	1.61	59.20	1.11	40.80	
9-*a*	Saturated fabric..........	50.77	69.06	1.28	0.63	49.20	0.65	50.80	
9-*b*	" "	54.70	41.15	1.28	0.57	44.50	0.71	55.50	
9-*c*	" "	62.50	63.41	1.38	0.86	62.30	0.52	37.70	
10-*a*	Saturated burlap	61.00	60.50	2.31	1.41	61.04	0.90	38.96	80
10-*b*	" felt..........	58.25	73.80	2.40	1.40	58.33	1.00	41.66	28

The felts are generally composed of wood or vegetable fiber, with a little wool in some cases, and in one case, asbestos fiber.

As to finish, there seem to be generally two kinds of these felts: 1st, the felt simply saturated with the asphalt compound, with what might be called a rough finish; 2d, a saturated and coated felt with the finish smooth and hard. Of these, the first is much the cheaper.

Table 2 shows that though there are considerable differences in the quantity of bitumen per square foot, expressed in the form of a percentage, the actual differences are not very great.

The fabrics alluded to are of two kinds: 1st, burlap; 2d, some kind of closely-woven cotton material. These are saturated with the asphalt compound during manufacture. Burlap should never be used in an untreated state on the work, as the raw material is not easily embedded in the hot asphalt in the field, and, further, it takes up water readily from the air. The temperature of the asphalts used should not be greater than 400°, as higher temperatures are likely to burn the burlap.

The figures, 1, 2, etc., in Table 2, represent different manufactures, and the letters, *a*, *b*, etc., refer to different samples.

APPENDIX C.

Rock Asphalts. (See Table 3.)

The only rock asphalts here treated are the imported materials from Switzerland, Germany, and Sicily. The rock, as imported from these localities, contains in itself the following quantities of bitumen.*

Ragusa, Sicily.	Seyssel, France.	Verwohle.	Sicula, Sicily.	Neuchâtel, Switzerland.	Mons.
9.9%	5.9%	7.5%	10.2%	9.1%	8.9%

This rock is ground and fluxed with additional asphalt, so that, when poured into cakes and shipped to the site of the work, the percentages of bitumen vary from 13 to 18%, and it is this material which is again melted, with the addition of about 10% of flux, and incorporated with grit or small pebbles to make the mastic.

The figures in Table 3 give the results of a number of analyses of two brands of asphalt mastic. In a few cases, the granularimetric analysis of the natural rock freed from the bitumen is given. It will be noted that this runs very high in the finest particles of the limestone, which is the natural rock impregnated in both kinds of asphalts tested.

TABLE 3.—Analysis of Rock Asphalts as Delivered on the Work.

No.	Total bitumen soluble in CS_2.	Percentage of total bitumen soluble in 88° P.E.	Percentage of insoluble organic matter.	Percentage of moisture.	Percentage of ash.	Granularimetric Analysis.					Remarks.
						Passing sieves:				Retained on:	
						No. 200.	No. 80.	No. 40.	No. 10.	No. 10.	
1-a	16.70	63.22	......	......	......	46.3	28.9	8.0	0.1	0.0	
1-b	17.40	68.96	......	......	......	51.4	24.6	6.1	0.5	......	
1-c	16.90	61.90	......	......	......	52.6	19.2	9.9	1.4	0.0	
1-d	14.47	......	10.95	0.12	74.46	......	......	......	......	......	
1-e	16.52	65.76	30.66	0.02	52.80	......	......	......	......	......	
1-f	17.93	62.80	3.01	0.21	78.85	......	......	......	......	......	
1-g	18.07	68.42	12.46	0.20	69.27	......	......	......	......	......	
1-h	11.06	67.03	39.32	0.02	49.60	......	......	......	......	......	
2-a	16.76	60.12	2.03	0.09	81.12	......	......	......	......	......	
2-b	17.70	59.48	......	......	......	59.8	13.0	9.2	0.3	0.0	
2-c	18.07	67.29	5.11	0.00	76.82	......	......	......	......	......	
2-d	15.90	......	6.7	......	......	......	......	......	......	......	
2-e	16.17	......	12.79	0.42	70.62	......	......	......	......	......	
2-f	13.42	93.82	12.21	0.09	74.28	......	......	......	......	......	
2-g	15.9	......	......	......	70.44	......	......	......	......	......	74.1% calc. rock powder.
2-h	13.27	......	16.14	0.15	......	......	......	......	......	......	
3-a	17.8	67.04	......	......	......	56.3	20.6	5.1	0.2	0.0	

Records, in a number of cases, of long life for these imported rock mastics, make them very desirable for water-proofing purposes under certain conditions. Cases of more than 20 years' use, with the material apparently in as good condition as it was when placed, are known.

* "The Modern Asphalt Pavement," by Clifford Richardson, M. Am. Soc. C. E., p. 253.

APPENDIX D.

MINERAL RESOURCES OF THE UNITED STATES FOR 1912.
PRODUCTION OF ASPHALT BY VARIETIES.

	Short tons.	Percentages.
Bitumen rock	53 041	11.7
Refined bitumen	22 852	5.1
Maltha	474	0.1
Wurtzilite (elaterite)	8 452	1.9
Gilsonite	31 478	7.0
Oil asphalt	333 213	74.2
Total	449 510	100.0

The total imports of asphalt in 1912 were 218 328 short tons.

APPENDIX E.

EXTRACTS, FROM AN ARTICLE ENTITLED "ASPHALT PAVING CEMENTS AND ROAD BINDERS", BY MR. J. W. HOWARD.*

Mr. Howard's article, though written with reference to roads and pavements, has much of interest in connection with the subject of the use of asphalts for water-proofing purposes, and the writer calls special attention to it in order that the various features may be elaborated by those interested in the subject, and ultimately form the basis of more specific data for water-proofing specifications.

What follows has been abstracted, briefly, with the object of showing some of the properties which are considered necessary in an asphalt, and without giving the laboratory methods, which are described in each case.

1. Adhesiveness, or Cementing Strength.—This is a self-evident property needed to cement together the particles of a mastic or the various layers of felt.

2. Water-proofness, or Freedom from Injury by Water.—This is important on account of the exposure of the materials to water moisture, either directly or through capillary action. The formulation of some test which can be made readily to determine this property is of special value, as it is well known that asphalts differ in the property of resisting the action of water.

3. Immutability, or Freedom from Deterioration on Exposure to Sun and Air.—This is important on account of the bad effects which may result from the heat of the sun, the action of the air, and other elements. The asphalt must retain its life and qualities. This test is made by heating, as called for in the specification presented, and noting the percentage of loss; also in examining the penetration before and after the heating.

4. Ductility.—Ductility is the ability to yield without breaking, and is an important property in asphalt used in solid-floor bridges. It is also necessary that the material should be able to contract from cold without cracking. The data for this property are embraced in the specifications, and have been studied more or less carefully. A reasonable and consistent ductility at various temperatures is a desired property.

5. Flexibility or Pliability.—Flexibility or pliability without brittleness is needed for practically the same reasons as are given for ductility, and because the asphalt should be able to yield without cracking. Very little data are available concerning this property.

6. Cohesiveness.—This is the property needed by an asphalt in order to prevent it from pulling apart or cracking within itself,

* *Engineering Record*, September 27th, 1913.

although it may adhere firmly to the materials which it is to bind together. This property is one that has not been studied in water-proofing materials, but is deserving of further care and study.

7. *Malleability.*—This quality is referred to by Mr. Howard, but is not required for water-proofing materials.

8. *Consistency.*—This is the right kind of hardness or softness necessary under various temperature changes. It is also known technically as penetration. The tests of the various asphalts and the methods of making the tests are given in Appendix A.

9. *Minimum Susceptibility to Softening or Stiffening During Extreme Weather Temperatures.*—All asphalts are softened by hot and stiffened by cold weather. Thus far it has been impossible to devise asphalts which will not do this. All good ones, however, do not become too soft or fluid in hot weather, or too stiff in cold weather. This is shown by the penetrations at freezing and at high temperatures. (See Appendix A.)

10. *Purity, or High Percentage of Bitumen.*—This quality is important, for several reasons. It excludes foreign matter, which is often injurious. Useless matter is not cementitious, and reduces the quantity of work that can be done with a given quantity of pure asphalt.

11. *Freedom from Injury by Heat.*—Because all asphalts must be heated sufficiently to apply in the work, it is important that they should not be injured by being heated and maintained at a fluid temperature for a reasonable time. The same test as for immutability may be used.

APPENDIX F.

PLANS OF DIFFERENT METHODS OF WATER-PROOFING.

Figs. 1 to 9 illustrate progress in water-proofing design, and should be considered and studied in the order in which they are numbered, as the sequence represents what are considered to be improvements in each case.

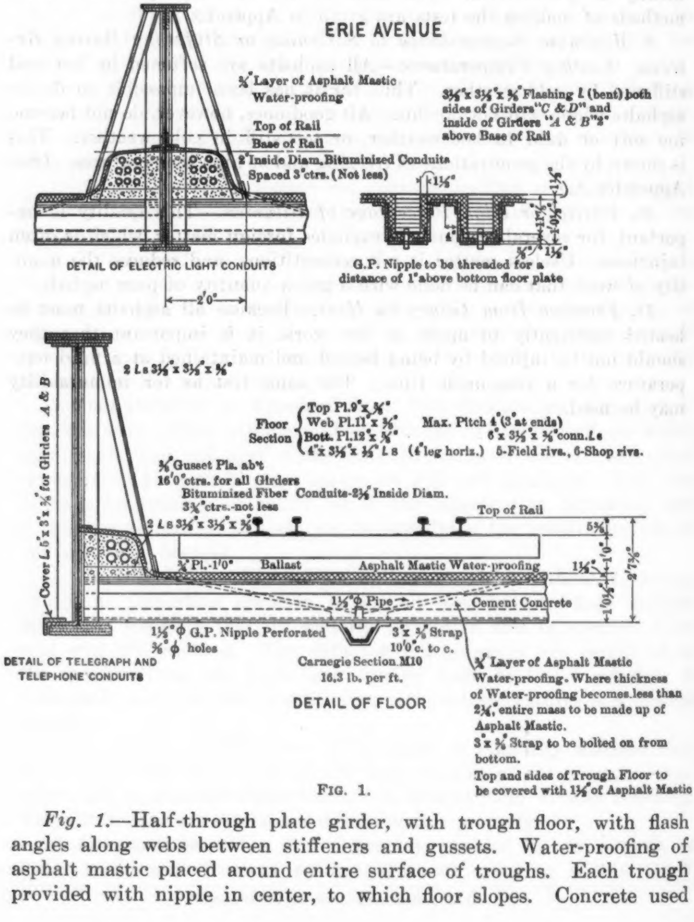
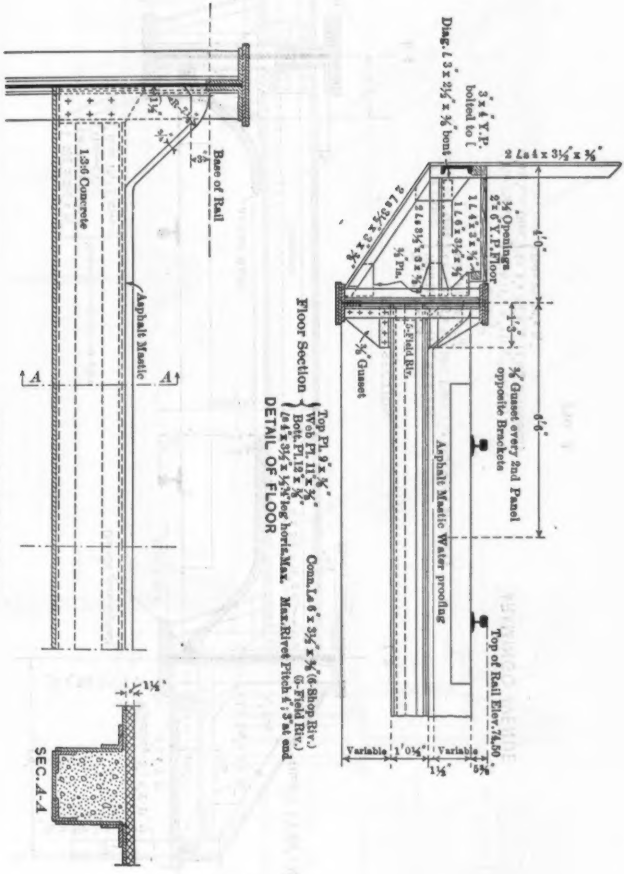


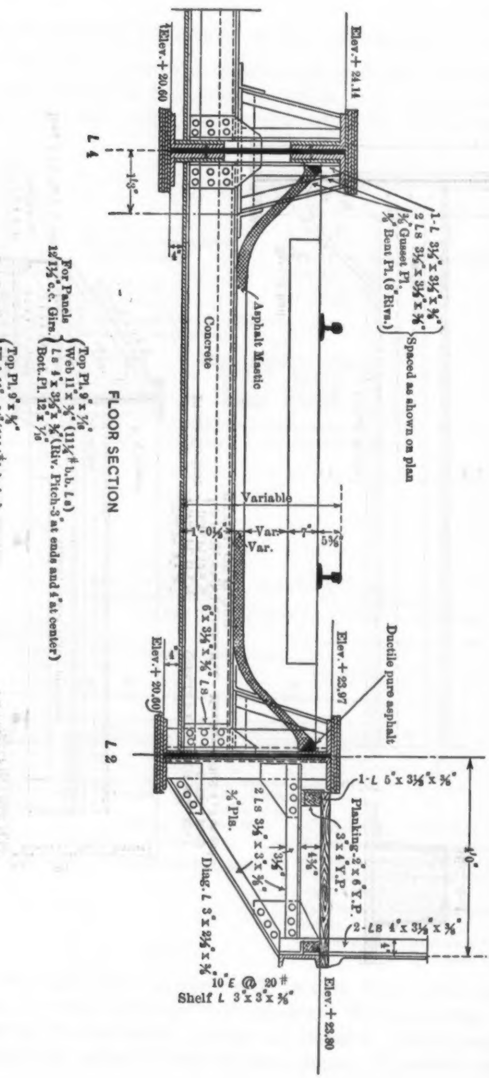
FIG. 1.

Fig. 1.—Half-through plate girder, with trough floor, with flash angles along webs between stiffeners and gussets. Water-proofing of asphalt mastic placed around entire surface of troughs. Each trough provided with nipple in center, to which floor slopes. Concrete used



MONTGOMERY AVENUE

FIG. 2.



FLOOR SECTION

FIG. 3.

ARAMINGO AVENUE

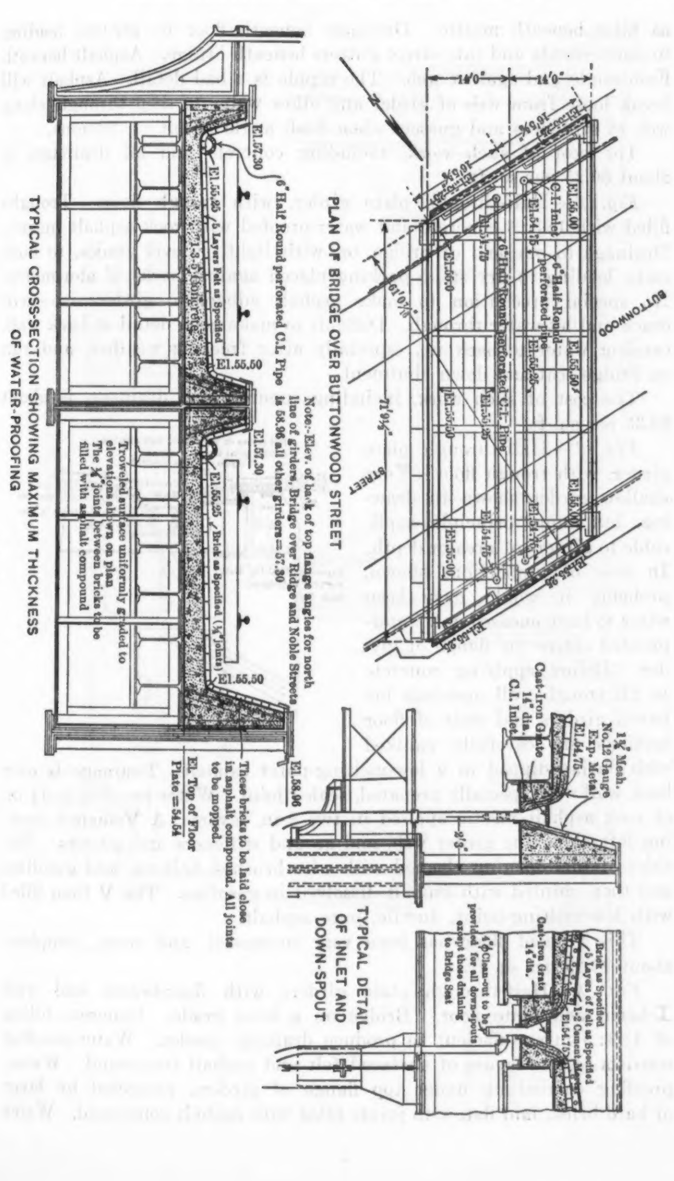

PLAN OF BRIDGE OVER BUTTONWOOD STREET
Note:-Elev. of back of top flange angles for north line of girders, Bridge over Ridge and Noble Streets is 58.96 all other girders=57.30
TYPICAL CROSS-SECTION SHOWING MAXIMUM THICKNESS OF WATER-PROOFING
TYPICAL DETAIL OF INLET AND DOWN-SPOUT
BUTTONWOOD
STREET
6" Half Round perforated C.I. Pipe
These bricks to be laid close in asphalt compound. All joints to be mopped.
Troweled surface uniformly graded to elevations shown on plan
The ¼" joints between bricks to be filled with asphalt compound
Cast-Iron Grate 14" dia.
C.I. Inlet
1½" Mesh. No.12 Gauge Exp. Metal
El. Top of Floor Plate=54.64

FIG. 4.

as filler beneath mastic. Drainage beneath floor by gutters leading to down-spouts and into street gutters beneath bridge. Asphalt beneath flash angle and against web. The nipple is a bad detail. Asphalt will break loose from web of girder and allow water to seep through along web at stiffeners and gussets when flash angle is not continuous.

The cost of such work, including concrete and all drainage, is about $0.25 per sq. ft.

Fig. 2.—Half-through plate girder, with trough floor. Troughs filled with 1 : 3 : 6 concrete and water-proofed with rock asphalt mastic. Drainage to one end of bridge, or, with light or level grades, to both ends, leading to dry stone packing placed against backs of abutments. No special precaution to make asphalt adhere to girder web—will crack and let water through. Difficult to make good detail at back wall, causing water to back up, especially after freezing weather, and run on bridge seat and down abutment.

The cost of such work, including concrete and drainage, is about $0.35 per sq. ft.

Fig. 3.—Half-through plate girder, with trough floor. Very shallow girder shown in drawing, but same principle applicable to girders of greater depth. In case of proportions shown, probably it would have been wiser to have encased and water-proofed entire top flange of girder. Before applying concrete to fill troughs, all openings between girders and ends of floor sections are carefully caulked with burlap dipped in a low-melting-point asphalt. Drainage is over back wall with specially prepared, sealed joint. Water-proofing is 1½ in. of rock asphalt mastic applied in two ¾-in. layers. A V-shaped opening left along the girder web, and around stiffeners and gussets. The sides of this opening cleaned with wire brushes, bellows, and gasoline, and then painted with asphalt dissolved in gasoline. The V then filled with low-melting-point, ductile, pure asphalt.

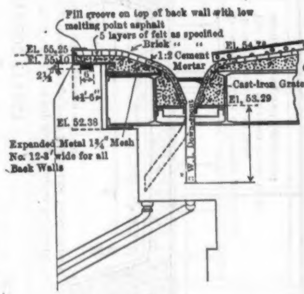


FIG. 5.

This type of floor has been very successful, and costs, complete, about $0.45 per sq. ft.

Fig. 4.—Half-through plate girders with floor-beams and with I-beam and plate floor. Bridge on a level grade. Concrete filling of 1 : 3 : 5 mix, sufficient to produce drainage grades. Water-proofing consists of five layers of surfaced felt and asphalt compound. Water-proofing carried up under top flange of girders, protected by layer of hard brick, laid flat, with joints filled with asphalt compound. Water

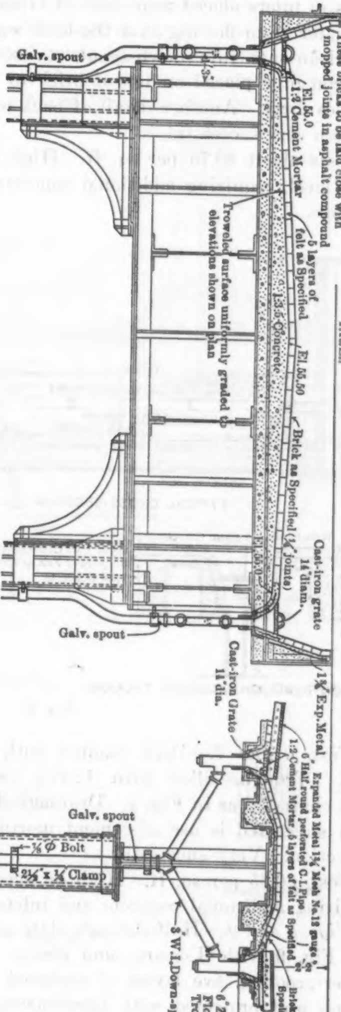

These bricks to be laid close with mopped joints in asphalt compound
Galv. spout
El. 55.0
1:2 Cement Mortar
5 layers of felt as Specified
1:3:5 Concrete
Troweled surface uniformly graded to elevations shown on plan
T.R.El.
El. 55.50
Brick as Specified (¼" joints)
Cast-iron grate 14" diam.
Cast-iron Grate 11" dia.
1¾" Exp. Metal
Expanded Metal 1¾" Mesh No. 18 gauge
6" Half-round perforated C.I. Pipe
1:2 Cement Mortar, 5 layers of felt as Specified
Brick as Specified
Galv. spout
⅞" Φ Bolt
2½" x ¼" Clamp
8" W.I. Down-spout Galv.
6" Z-Bar Trough Floor
Troughs 1'9" c.c.
3'1½"

FIG. 6.

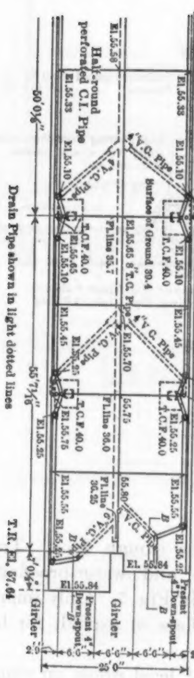

Half-round perforated C.I. Pipe
Drain Pipe shown in light dotted lines
Surface of Ground 30.4
T.R. El. 57.64
Present Down-spout
Girder
25'0"

FIG. 7.

drains to inlets placed near ends of bridge and graded so as to prevent any water from flowing over the back walls. Drainage through grates and cleân-outs into down-spouts to sewer. Half-round, perforated, cast-iron pipe placed on top of brick to assist flow of water if ballast becomes dirty. Another detail of back-wall drainage, shown in Fig. 5, has been very successful.

Costs about $0.75 per sq. ft. High on account of grade of track being level, requiring additional concrete and inlets to provide proper drainage.

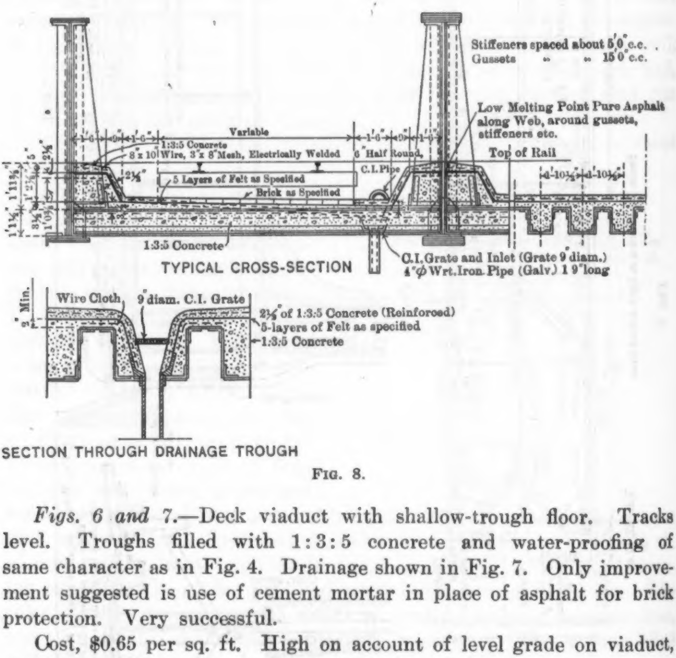


FIG. 8.

Figs. 6 and 7.—Deck viaduct with shallow-trough floor. Tracks level. Troughs filled with 1:3:5 concrete and water-proofing of same character as in Fig. 4. Drainage shown in Fig. 7. Only improvement suggested is use of cement mortar in place of asphalt for brick protection. Very successful.

Cost, $0.65 per sq. ft. High on account of level grade on viaduct, requiring additional concrete and inlets to provide proper drainage.

Figs. 8 and 9.—Half-through plate girders. Fig. 8 with trough floor and Fig. 9 with I-beams and plates. Filling with 1:3:5 concrete. Water-proofing, five layers of surfaced asphalt felt with asphalt compound, and protected with hard-burned brick in 1:3 cement mortar on flat floor and 1:3:5 concrete, reinforced with No. 8 and No. 10 wire, 3 by 8-in. mesh, electrically welded in gutters and over haunching. Pocket of low-melting-point, pure asphalt, carefully put in as

seal against girders and around stiffeners and gussets before protection is placed. Drainage to inlets. No water going over back walls. Water removed from copings by special inlets built in masonry.

Under contract and not completed at present time. Cost about $0.62 to $0.80 per sq. ft., including all drainage details.

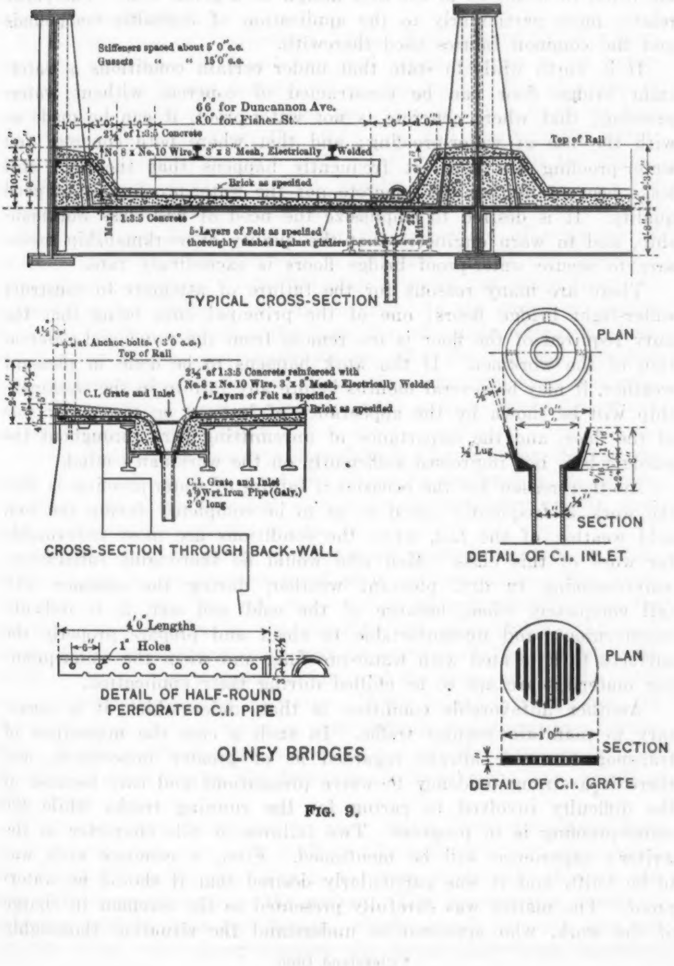


FIG. 9.

DISCUSSION

Mr. Himes.

ALBERT J. HIMES,* M. AM. SOC. C. E. (by letter).—Mr. Wagner's paper is particularly useful, because the water-proofing of bridge floors has not yet crystallized into standard practice, and engineers are often in doubt as to the best design in a given case. The paper relates more particularly to the application of asphaltic compounds and the common fabrics used therewith.

It is worth while to state that under certain conditions a water-tight bridge floor can be constructed of concrete without water-proofing; that where concrete is not water-proof, it can be made so with the use of water-proofing; and that where both concrete and water-proofing are used, it frequently happens that the floor still leaks because neither the concrete nor the water-proofing is of good quality. It is desired to emphasize the need of first-class workmanship, and to warn engineers that the quality of workmanship necessary to secure water-proof bridge floors is exceedingly rare.

There are many reasons for the failure of attempts to construct water-tight bridge floors; one of the principal ones being that the duty required of the floor is too remote from the mind and observation of the workmen. If the work happens to be done in pleasant weather, it may be several months before any failure in the workmanship will be shown by the appearance of leakage on the under side of the floor, and the importance of unremitting care throughout the whole job is not impressed sufficiently on the workman's mind.

Another reason for the occasional failure of water-proofing is that the work is frequently timed so as to be completed during the wet, cold weather of the fall, when the conditions are most unfavorable for work of this class. Men who would do thoroughly satisfactory water-proofing in dry, pleasant weather, during the summer, will fail completely when, because of the cold and wet, it is difficult, inconvenient, and uncomfortable to clean and prepare properly the surfaces to be coated with water-proofing, and when the water-proofing materials are apt to be chilled during their application.

Another unfavorable condition is that under which it is necessary to maintain regular traffic. In such a case the necessities of transportation are always regarded as of greater importance, and there is a strong tendency to waive precautions and care because of the difficulty involved in caring for the running tracks while the water-proofing is in progress. Two failures of this character in the writer's experience will be mentioned. First, a concrete arch was to be built, and it was particularly desired that it should be water-proof. The matter was carefully presented to the foreman in charge of the work, who appeared to understand the situation thoroughly

* Cleveland, Ohio.

Mr. Himes.

and, therefore, was relied on to make water-tight concrete, no water-proofing being used. Soon after the removal of the forms, various leaks were discovered, much to the chagrin of those in charge of the work.

At a later date, a similar arch was to be constructed under the same conditions, but, with the bad example for reference, the necessary care and attention were not wanting, and the second arch proved to be entirely satisfactory.

In another case, a double-track bridge floor, consisting of a concrete slab supported on floor-beams and stringers, was to be water-proofed while traffic was maintained on one track. The plan was to water-proof one-half of the bridge first, then to lay a track on that side, remove the track from the other side, and complete the water-proofing. In order to secure a tight joint on the axis of the bridge, the fabric was allowed to project loosely about 12 in. beyond the center line, thus providing sufficient lap.

The work was being done under the inspection of employes of another road who were disposed to be very critical and exacting. Instructions were given to the foreman to carry out their directions to the letter, and without any controversy. In the course of events, the inspector demanded that the loose flap of fabric which had been provided for the lap-joint should be cut off. The foreman complied with his directions, and the result was that, when the work was complete, a longitudinal seam existed along the axis of the bridge, which was anything but water-tight.

In the design of a concrete floor-slab, it should always be remembered that cracks will be produced either by contraction or flexure, and joints should be provided with sufficient frequency and in such locations as to forestall the cracks.

A square street crossing, without curb supports and about 50 ft. in length, can be made water-proof with concrete alone. The introduction of curb supports will cause points of contraflexure over the supports, where cracks in the concrete are unavoidable. At such places, sheets of lead or copper may be inserted in the concrete with a fold to provide for a slight motion, and the floor can thus be made secure against the passage of water. If the bridge is on a skew, the design of such a joint becomes very difficult and sometimes impossible. Where water-proofing is used to prevent the water from passing through such cracks, the fabric is likely to tear, thus defeating its purpose.

The writer has endeavored to carry water from the bridge floor over the back walls into wells built to take care of it, but agrees with Mr. Wagner that the best way is to carry it directly through the floor with down-spouts into the sewers. The down-spouts should be encircled with at least two flanges, which should be built into the concrete floor, not placed against the upper and lower surfaces.

Mr. Himes. Flanges thus built into the concrete will effectually stop the passage of water along the pipe.

The writer does not agree with Mr. Wagner in his preference for a pocket of mastic along the web of the girder. It is better to rivet a flange angle along the web and to use malleable cast flanges around the stiffeners and gusset-plates. The water-proofing can then be carried up the side of the girder under the flange angle, and the joint will thus be sealed absolutely against the passage of water.

Turning to the specifications, comments are made as follows:

2.—Grades of 1% on the bridge floor are very difficult to secure, and may require an excessive weight of concrete. A thickness of concrete of $2\frac{1}{2}$ in. is not sufficient to cover permanently the steel-work of the floor.

3.—The inlets should be provided with flanges, as previously described.

4.—Connecting longitudinal girders rigidly over a column causes motion in the bridge under the passage of trains. This motion in time develops looseness of joints, rust, and wear. Whenever it is desirable that girders should be thus connected, there should be such surplus of stiffness as to do away quite completely with the deflection.

10.—The protection of water-proofing with a layer of brick has been found in some cases to result in tearing the fabric of the water-proofing, because of the motion of the brick. This method, therefore, is not always satisfactory.

11.—Any pipe lying on the bridge floor is very likely to become clogged with cinders, and would not be recommended by the writer. He would prefer to allow the water to flow freely on the surface of the bridge and to provide as many down-spouts as possible.

13.—It would be better to use 1 : 2 : 4 concrete and do away with the $\frac{1}{2}$-in. coating of cement mortar on the top. If the concrete is well made, the $\frac{1}{2}$-in. coating is unnecessary, and it very often happens that the coating is placed so that it soon separates from the concrete below.

39.—Where water-proofing is to be used, a special effort should be made in planning the work to time its completion so that the water-proofing can be done during pleasant summer weather.

41.—The pocket of asphalt provided is inferior to the use of flange angles and malleable castings at the stiffeners.

44.—The writer would inquire whether any one has found a thin layer of sand on top of the water-proofing, covered by a thin layer of concrete to protect the latter against injury by picks and tamping bars, to be a better protection to the water-proofing than the layer of bricks described.

52.—This clause of the specification is probably intended to insure that the water-proofing will not be done in unfavorable weather. **Mr. Himes.**

54.—This clause is very general. A detailed description of the care to be exercised would be more useful.

In Appendices A and C, descriptions are given of certain tests to determine the quality of the asphalt. The writer would inquire whether the careful performance of these tests would insure a satisfactory quality of asphalt with the same certainty that one feels in the testing of cement or steel.

In Fig. 1, Appendix F, it is to be noted that the gutter projects below the bottom of the girder flanges and, therefore, diminishes the clearance beneath the bridge. This is a condition which is not permissible in city streets. The expense of securing the necessary clearance is too great.

It is a satisfaction to note the passing of the excessively shallow floor without ballast. The noise from these floors is intolerable. Henceforth, only ballasted bridges should be built, and the writer is inclined to think that the use of sand for ballast may be found more satisfactory than stone.

WILLIAM S. BABCOCK,* ASSOC. M. AM. SOC. C. E. (by letter).—The methods suggested by Mr. Wagner for the water-proofing of steel-floor railroad bridges are of the very best and most approved, but the paper is misleading as to the materials generally used, inasmuch as Mr. Wagner makes no reference to coal-tar pitch or tar-saturated felt. **Mr. Babcock.**

Within the last 6 years, there have been approximately 1 000 bridges water-proofed with these materials, and in the majority of cases, good reports have been received. The method used has been practically the same as that described in the author's "General Specifications", but the materials have been straight-run coal-tar pitch and a 14 to 15-lb. coal-tar saturated felt, with a core of felt reinforced with cotton drilling.

For this type of water-proofing, the following materials have been used: 4 plies of 14-lb. tar-saturated felt; 1 ply of reinforced cotton drilling felt; and 250 lb., per 100 sq. ft., of straight-run coal-tar pitch.

The protection for the water-proofing which has been used was similar to that described under "Protection of Water-Proofing" by Mr. Wagner, with the exception that coal-tar pitch was used in the mastic, instead of asphalt. The preparation for water-proofing, using coal-tar pitch, would be the same as that suggested in the paper.

Referring to the statement, "The specifications, as presented, have been the outgrowth of about eight years' experience * * *," the following might be of interest:

* New York City.

Mr. Babcock.

The earliest data the writer has regarding the inception of bridge water-proofing were obtained on the New York Central and Hudson River Railroad. About 1888, that Company began the water-proofing of trough bridges on the Park Avenue Viaduct, in New York City. This water-proofing consisted of a binder, in some cases of coal-tar pitch, and in other cases of asphalt. This was not at all satisfactory. From this arose the necessity of producing flat-surface bridges of concrete.

Table 1 gives some tests of water-proofing asphalts. The author does not state when or where these tests were made, nor whether the analyses refer to asphalts which gave good, bad, or indifferent results, which leads one to inquire as to the usefulness of the results in connection with this paper.

Appendix E contains extracts from an article by Mr. J. W. Howard entitled "Asphalt Paving Cements and Road Binders", and the author states that, although written in reference to roads and pavements, it has much of interest in connection with the subject of asphalts for water-proofing purposes. He also states that these features may be elaborated by those interested in the subject, and ultimately form the basis of more specific data for water-proofing specifications.

Without definitely taking exception to Mr. Howard's statements, as quoted by the author, the writer desires to call to the attention of engineers the paper by Mr. A. W. Dow on "Relation Between Some Physical Properties of Bitumens and Oils."* In this paper, Mr. Dow sets forth some theories which would explain the apparent peculiarities of different types of bitumen, and his theories have not since been controverted successfully; in the writer's judgment, they may well form the basis of a study of the properties necessary in a bitumen for water-proofing purposes.

Mr. Jones.

JONATHAN JONES,† ASSOC. M. AM. SOC. C. E. (by letter).—During 1914, the City of Philadelphia constructed a five-track bridge, of 90 ft. clear span, to carry the Philadelphia and Reading Railway tracks. The floor design and details were as shown in Fig. 9, and the specification for water-proofing was an earlier form of that proposed in this paper. In this earlier form, considerable latitude was given to the engineer, as to certain details, so that in effect the bridge may be said to have been water-proofed according to the specification presented in the paper.

The price paid to the sub-contractor for water-proofing was about 47 cents per sq. ft. of horizontal projection, including all construction above the steel floor-plate. This is a lower price than would usually be obtained.

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* *Proceedings*, Am. Soc. for Testing Materials, Vol. VI, p. 497.

† Philadelphia, Pa.

The work was carefully followed up, and not a move was made except under the eye of the inspector; the design and specification, therefore, must take the blame if good results are not obtained in this case; at present they appear to be entirely satisfactory. The bridge has been under traffic for several months, and has been subjected to all kinds of weather conditions, including melting snow and ice, and it is perfectly water-tight. Mr. Jones.

In carrying out this work and observing its behavior, several points have occurred to the writer as needing further consideration before the present design and specification may be considered as standard.

It is not clear that the ½-in. steel plate used on the floor-beams is superior to a non-reinforced concrete slab, haunched down on the beams and girders. Such a slab would preclude the necessity of painting the beams, and as its soffit can be kept parallel to the surface of the water-proofing, it will be a tougher body than the minimum of 2½ in. shown in Fig. 9. It may be said that the steel plate is one additional water-proof membrane, but the plain slab will neither crack under traffic nor be pervious if properly poured, and it will be tighter along the girders, where trouble may be expected if it occurs at all. The saving in cost of the slab should pay for the extra metal in the girders due to increased dead load, and, if so, it is known that, of two bridges costing equal amounts, the heavier is, other things being equal, the better purchase.

It may prove necessary to insulate the 4-in. drain pipe and carry it into the abutment beyond the possibility of freezing. On sunny, winter days, the snow and ice on the bridge deck will melt and trickle down the drain, but as this is shaded and cold, the tricklings will freeze until the pipe is choked. Another rain following this occurrence, of course, will create a condition not cared for by the design.

There is considerable wasted dead load in the concrete haunches, and it would be well to consider burying vitrified sewer pipes, with end closures, in these haunches to save a little in first cost and to reduce the weight. There would still be ample bulk of concrete for the duty it performs.

Instead of caulking with asphalt, as specified in Paragraph 12, good results have been obtained by using cement mortar. These bridges are usually painted a light tint on the under side, to brighten the street, and when the asphalt used in caulking runs down through the joints, as it inevitably will, the appearance from below is very unsatisfactory.

Instead of the cement mortar coat specified in Paragraph 13, the writer advocates a granolithic mortar, as the former will always hair-crack on drying out. An expert sidewalk finisher is needed for this part of the work, as the 1% grade specified is only the minimum

Mr. Jones. for carrying off the water, and no chance should be taken of losing any part of this grade by irregular finishing.

Incidentally, the last sentence of Paragraph 46 may be questioned as having been carried over from earlier designs and being inapplicable to those of Figs. 8 and 9. The writer believes it would be of value if Mr. Wagner would explain his preference for a protective covering partly of brick and partly of reinforced concrete, rather than wholly of one or the other; also his preference for bedding the brick in cement mortar rather than in asphalt. It would seem that the cement mortar will unite the brick rigidly to the felt, and when the bricks crack apart under vibration, the cracks will be communicated to the felt; but, with a yielding bond, such as asphalt, this would not occur.

All such water-proofing systems must be considered as tentative until they have had the test of years, because of the possibility of deterioration within themselves, as well as because of the clogging of the ballast, which is the curse of all such systems. Mr. Wagner, however, may be immediately congratulated on having gotten away from metal flashings as his main reliance. Those flashings were a torment to the drafting-room and bridge shop. The writer knows of one large contract on which the shop cost was actually doubled by the everlasting patching and fitting of those flashings around the stiffeners. One great advantage of the design proposed by Mr. Wagner is that this fitting is done in the field with plastic materials, and not with metal.

Mr. Allen. J. LEE ALLEN,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer has been for years interested in showing the permanency of the results obtained by the use of European asphaltic limestone rock mastic.

The author, after personal examination, shows some of the excellent results obtained from the use of European natural asphaltic rock mastic as a water-proofing medium, in which examinations the asphaltic rock mastic was found in splendid condition after 20 years' service. This perfection can be secured to-day, as the same materials are available, and, to make sure of their validity, the following clause should be added to the general specifications:

"The asphaltic rock mastic blocks shall be imported European asphaltic rock mastic, brought to the place of work in original blocks with the brand stamped thereon. The mastic blocks shall be shipped to the site of the work on a through bill of lading."

This clause would eliminate all the so-called mastics manufactured in the United States, which are made without the use of the natural European asphaltic limestone, and in which the manufacturers attempt to reproduce, in a few moments, what it has taken Nature ages to accomplish, and this is the reason for its failure. The great value of

* Chicago, Ill.

Mr. Allen.

the European asphaltic rock mastic lies in the close relation between the bitumen and the mineral portion of the material, the former thoroughly impregnating the latter and resulting in a dense and stiff mass, although still very elastic and not susceptible to climatic changes, as a result of the soft condition and highly cementitious nature of the bitumen.

W. H. Finley, M. Am. Soc. C. E., refers to bridges* water-proofed in 1905 by the use of imported asphaltic rock mastic which have proved very satisfactory.

The writer, during his experience and observations of the result of the use of European asphaltic rock mastic for water-proofing purposes, other than for bridges, has found the materials perfect and still in daily service after 30 years of continuous use.

Mr. Woodruff.

Glenn B. Woodruff,† Jun. Am. Soc. C. E. (by letter).—There is probably no question more studied by the present-day railway bridge designer than that of water-proofing solid-floor bridges satisfactorily. At the same time, there is little published information relative to the experience of different engineers on this subject, and therefore the author deserves the thanks of those working on the problem for giving the results of his experiences. In some respects, however, it seems that there is room for a difference of opinion.

One of the most important problems is that of drainage, which resolves itself into the proposition of removing the water from the bridge in the shortest possible time and in such a manner that there is the least chance of any moisture getting between the water-proofing and the steel. The writer believes that in structures of moderate spans the floor should be graded so as to force the water over the back wall, where it may be cared for by building a concrete gutter or even by a hollow tile drain lying on the sub-grade. If the attempt be made to carry the water through the bridge floor, it is a difficult matter to flash the down-spout so that no water will find its way under the water-proofing. Also, a screen small enough to hold back the smaller pieces of ballast is very likely to become clogged. Even where a means of cleaning this drain is provided, it is more than likely that it will not receive attention until the water makes its appearance at some undesired point.

The author's detail of substituting a pocket of asphalt mastic for the customary flash angle has advantages until the asphalt hardens or otherwise deteriorates. It may be questioned whether a flash angle above this asphalt pocket would not form sufficient additional protection to warrant its use.

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* "Waterproofing of Engineering Structures," *Journal*, Western Soc. of Engrs., Vol. XVII, p. 545.

† South Bethlehem, Pa.

Mr. Woodruff.

The writer objects to placing the water-proofing membrane directly on the steelwork. In case this membrane grows hard and cracks, if it becomes damaged in any way, or if there is a slight air space between the steel and the membrane so that there is a possibility of condensation, water comes in contact with the steel under conditions favorable for corrosion and in a place where it cannot be detected. For this reason it is believed that a minimum of 3 or 4 in. of concrete, well reinforced, should be placed between the steel and the water-proofing. This concrete should be as dense as possible, and, in each case, experiments should be made with the aggregate to determine the proper mixture. The addition of a small percentage of hydrated lime is advisable.

Following out this scheme, the writer believes that a reinforced concrete slab carried by transverse rolled beams is, in many respects, at least, more satisfactory than the trough type of floor. In different cases, the writer has made comparisons of the cost of the two types and generally has decided in favor of the former. The same scheme may also be used to advantage in double-track bridges where the conditions demand a very shallow floor.

Mr. Ray.

G. J. Ray,* M. Am. Soc. C. E.—This paper is particularly valuable to those who have to do with the design and construction of solid steel-floor railway bridges. The author's exposition of "Preparation for Water-Proofing" and "Materials and Application" will be found especially valuable in the selection of a suitable water-proofing material, and its proper application, whether or not the structure to be water-proofed is similar in design to those described in Appendix F.

The speaker has found a material lack of reliable data and needed instructions for the design and application of water-proofing on various kinds of railroad bridges, outside of those recommended by the numerous firms selling such materials. In fact, one is apt to discredit much that is claimed for various water-proofing materials when one finds that nearly all water-proofing salesmen are inclined to run down the materials sold by their competitors.

The author's contribution, therefore, is especially valuable, as it outlines the subject from a practical viewpoint, and the specifications are based on actual experience and results attained. Again, he has not confined the paper to one particular method of application.

The title of the paper, as well as the first paragraph of the synopsis, limits the consideration to "bridges with solid steel floors". Again, all the floor details given in Appendix F are of the steel-trough or steel-plate floor design. The speaker assumes, however, that it is in order to discuss the design and water-proofing of steel bridges of any other type with solid floor, which answers the same purpose as those described.

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* Hoboken, N. J.

The first of the solid steel-floor bridges, with which the speaker had any experience, was built between 1903 and 1906. The floors of the deck bridges were made of channels, with the flanges up, and riveted together. The floors of the through bridges were of the ordinary trough type. With both through and deck bridges enough concrete was placed to provide good drainage. In most cases, the water was run off the bridges at one end, as they were on heavy grades. Plates were turned down over the back wall and, although fairly good drainage was provided, this construction did not prevent the water from finding its way through the back wall and down over the face of the abutment. No water-proofing was placed on the deck, and it was soon found that the ballast would have to be removed, and water-proofing applied. The water found its way down through numerous cracks formed in the concrete on top of the solid steel floor. Mr. Ray.

Five-ply felt, with brick protection, was used later when these structures were water-proofed. This water-proofing has eliminated the leakage through the floor system, but there is still some trouble with seepage through the back wall and over the abutment. Where this trouble was serious, the nuisance, to a great extent, has been eliminated by constructing a copper gutter along the abutment under the bridge seat.

In 1907 there was built, on the Delaware, Lackawanna and Western Railroad, a six-track deck-girder bridge, with a reinforced concrete floor, 84 ft. wide and 349 ft. long. The bridge is at the neck of a yard with numerous cross-overs and slip switches on the bridge, so that the floor had to be sufficiently strong to permit the maximum loading at all points. A reinforced concrete deck, 1 ft. thick, was placed directly on top of the girders. Drainage outlets through this floor were provided at low points in each 814 sq. ft. of surface. As the bridge was rebuilt under traffic, the floor could be built for only two tracks at a time. A ridge in the concrete was made at the construction joints to prevent leakage, but no water-proofing was applied. Although this floor was built and put in service 7 years ago, it is in good condition to-day, and no leakage has occurred. Other bridges have since been built in the same manner and with equally good results. In one case, in 1908, the speaker built a deck-girder bridge, 91 ft. wide, with a clear span of 60 ft., over a city street, with a reinforced concrete floor, without the application of water-proofing. The floor is in perfect condition to-day, and has never leaked a drop; neither has any water found its way over the back wall to the bridge seat. The floor is 16 in. at the center and 13 in. at the ends, and the slab extends past the back wall, with a groove in the underside to intercept the seepage.

At first, the speaker had some doubt as to the success of the last-mentioned construction over city streets, as there is no question about

Mr. Ray. the destructive tendency of water in passing through concrete. Therefore, nearly all the solid-floor bridges on his road have been water-proofed, regardless of the type of construction. During the past five years, his company has built deck-girder bridges over city streets with concrete slab floors, similar to those described in the paper; also numerous through-girder ballast-floor bridges with reinforced concrete slabs over the floor system, and also with the entire floor system—or bridges as a whole—encased in concrete. As a rule, the concrete for one or more tracks is poured complete in one operation, thus avoiding construction cracks at points where leakage is likely to occur. Where construction joints are necessary, copper flashing strips can be built into the concrete on each side of the joint, thus eliminating the leakage. These bridges, as a rule, have been water-proofed with five-ply felt, with brick protection, or with treated burlap and asbestos felt with asphalt mastic protection. There have been only a few leaks with either method, and up to date no trouble has been experienced from leakage through the sliding joint between the floor slab and the back wall. A double layer of treated felt or cotton drill has been placed in this joint with good results, and in all cases great care has been taken to extend the water-proofing at least 18 in. down over the back wall, and provide a good drainage system to carry off the water.

The only indication of water passing through the floor system has been a very slight seepage in a few places along the girders where the steel was painted prior to the construction of the concrete floor. The speaker believes that the surest way to prevent this trouble is to eliminate painting of steelwork which is to be covered with concrete.

With long structures, the author's method of taking care of the drainage should produce good results where the drain pipes can be connected directly to the sewer, without traps, so that the pipes will not freeze during cold weather. With short bridges over city streets, it is difficult to maintain longitudinal drains under bridge floors, and down-spouts at abutments to street sewers. With a temperature of about 32° Fahr., ice and snow will melt on the bridge floor and freeze in the leaders and drains under the bridge. This difficulty is not so great where columns are placed on the curb line, and down-spouts can be carried directly to the sewer by vertical pipes. It will be found, however, that, in cold climates, such drains are constantly freezing, and are a source of expense to maintain.

Where columns are permissible on the curb line, the speaker has used steel construction encased in concrete for the long span over the paved portion of the street and reinforced concrete slabs over the sidewalks. This construction permits of a perfect job of water-proofing, and eliminates any possibility of water getting down the face of the abutment from seepage through the back wall. In such cases, the columns have been constructed of concrete; the depth required

from the top of the rail to the under-clearance does not exceed 4 ft. 7 in., for spans up to 50 ft. centers. This type of construction, as a whole, is not unreasonably expensive. The encased steelwork is costly, but the slabs over the sidewalks are economical, on account of the short spans, and the thickness of the abutments can be reduced materially. Standard track centers can be maintained by keeping the center girder shallow. Painting is entirely eliminated, and the design can be made ornate enough to please the most skeptical critic. Mr. Ray.

On eight through-girder bridges recently constructed, the cost of the concrete floor, including reinforcement and encasement of I-beams, averaged 35.6 cents per sq. ft. of floor space. These bridges varied in length, from center to center of bearings, from 53 ft. 8 in., to 120 ft. 7 in., and the depth from top of rail to under-clearance of girders varied from 3 ft. 8 in., to 4 ft. All water-proofing was done with three-ply treated burlap and asbestos felt, protected with asphalt mastic, $1\frac{1}{2}$ in. thick. The average cost of water-proofing was 28 cents per sq. ft., thus making the entire cost of the floor system, 63.6 cents per sq. ft., exclusive of the structural steel. The steel structures were no heavier than would have been required by the method shown by Fig. 9.

In conclusion, it is the speaker's opinion that:

1st.—The specifications for materials and method of applying them, as set forth by the author, should obtain good results.

2d.—The specifications for details of construction for the types of structures described in Appendix F, in general, should insure workmanlike finish and secure good results, especially where the drainage system can be connected directly to the sewers without intervening traps.

3d.—With through-girder construction, and a depth of from 3 ft. 8 in., to 4 ft., from top of rail to under-clearance, a floor system can be designed which will permit of encasement in concrete, thus eliminating painting, and insuring a water-tight job at less expense than the cost of the floor shown by Fig. 9.

4th.—With deck-plate girder bridges, a water-tight reinforced concrete floor, without water-proofing, can be constructed which will not exceed 12 in. in thickness.

5th.—Where water-proofing is to be applied to any type of solid-floor bridge, the kind of materials to be used should be determined before the details of construction are fixed.

J. B. W. GARDINER,* ESQ.—Before making any comment on this paper, the speaker wishes to express both to Mr. Wagner and to the Society his appreciation of their courtesy in granting him the privilege of participating in the discussion of a most interesting subject. Mr. Gardiner.

* New York City.

Mr. Gardiner. With a keen and thorough knowledge of the conditions to which a water-proofing on the floor of a steel bridge is subjected, Mr. Wagner has given a very complete specification which the asphalt should fulfill. Not that any asphalt which will fill this specification will necessarily prove an efficient water-proofing material; but probably it will. However, if it cannot fill this specification, it most certainly will not prove efficient. It is thought, however, that it should be a part of the specification that the asphalt should be unaffected by the alkalies in cement mortar or the acid-impregnated drippings that come through cinders, and that it should adhere strongly to the concrete and to the membrane in conjunction with which it is used.

The question of a proper water-proofing membrane, however, is deserving of more extended discussion than Mr. Wagner has given it. Although it is unquestionably true that no membrane is of itself a water-proofing agent, the function which it performs in a water-proofing system is fully as important as that performed by the plying cement. They are entirely interdependent. The membrane is a binder—and a binder only—and as such its function is to hold in place the asphalt which is expected to shed the water and carry it to the drains. In the course of application, the membrane and the plying cement are, or should be, firmly cemented together so that they are practically one. Consequently, any movement of one part of the system occurs simultaneously in the other. Therefore, in so far as they are applicable, the specifications for the membrane should demand the same qualities as those for the plying cement. To illustrate: Mr. Wagner's asphalt specification requires ductility through a wide range of temperatures, which means that it must remain firm and elastic under extreme changes of weather. This quality in asphalt is essential, first, that it may not be shattered by the shock of impact when a moving train first passes on the bridge, or by the vibration of the entire structure due to the moving load; and secondly, the asphalt must yield or stretch to meet both the slight deflection which occurs as the center of mass shifts with the load and the movement in the concrete base incident to temperature changes. Expansion joints, it is true, eventually take care of this movement, but they do not do so immediately. By this, the speaker means that the movement is taking place throughout the entire length of the slab and is merely summed up at the expansion joints. That is, the movement which occurs at the expansion joint is a summation of an indefinite number of smaller movements which take place between the center of the slab and the expansion joint. If then we encase the ductile asphalt between binders of a rigid inelastic membrane, we have sacrificed all the advantages derived from this ductility. To be logical, therefore, we should insist that the elasticity of the membrane be equal to the ductility of the asphalt.

Another point to be considered is the presence of oils which show more than a certain percentage of volatility when subjected to a temperature of 325° Fahr. for a specified time. Far fetched as it may seem, this test has a distinct bearing on membranes. In order that the membrane may have the maximum protection against rotting, it should be thoroughly saturated with a stable asphalt. Under no circumstances should bitumen solvents be used in this process. In many cases, water-proofing membranes are treated by immersing them in a bath of asphalt liquefied by the addition of petroleum residue, in the proportion of 1 part asphalt to 4 or 5 parts residue. This applies principally to felts and burlaps. The result is saturation, it is true, but saturation largely with the petroleum residue, which, being a bitumen solvent, will eventually re-act against the plying cement. The specifications, therefore, should require that the membrane be saturated with an asphalt which would comply with Mr. Wagner's specification. As Mr. Wagner has pointed out, flashing effectively against the web of a half-through girder is an extremely difficult problem, largely because of projecting stiffeners, gussets, knee braces, etc. This difficulty is not lessened by the use of a stiff non-flexible membrane which cannot be made to fit snugly against the web at all points. Flexibility would seem, therefore, to be an important quality of the membrane. This fact is largely responsible for the use, first, of burlap, and, later, of cotton fabric as water-proofing membranes. Mr. Gardiner.

A few words as to the relative merits of cotton, and of burlap, which is a jute fiber, might not be misplaced. Jute consists of the chemical compound of cellulose with lignin, to which some investigators have given the name, bastose. It is much more readily affected by the action of acids and alkalies than cotton fiber, which is 95% pure cellulose. The influence of moisture, and even of air, will also rot the jute fiber. These characteristics go far toward fixing the cause of the trouble that so many engineers have experienced with water-proofing of this type.

Two other points in connection with membranes deserve passing mention, namely, resistance to puncture, and tensile strength. The desirable resistance to puncture, measured in pounds per square inch, by a standard paper-testing machine, should be determined by the chief engineer. The minimum tensile strength should be sufficient to stand the test for elasticity, without fracture, with a reasonable factor of safety.

As to the method of application; there is one point mentioned by Mr. Wagner, the reason for which the speaker does not understand, and in which, so far as his knowledge of the subject goes, he cannot agree. That is, the requirement that the first layer of felt should not be cemented to the floor of the bridge. If there is merit in it, it would

Mr. Gardiner. seem logical to lay the protecting brick course in a sand cushion instead of cement mortar, so as to keep the water-proofing course separate and distinct from either the base or the protection. On the other hand, one of the cardinal principles of good application is the avoidance of bridges or spans, so that when weight is placed on the water-proofing it will not break because of lack of support. This can only be obviated by making the first layer of membrane conform absolutely to the surface; and this is possible only by cementing the first layer to the surface thoroughly at all points. The possibility of puncture from other causes is also much more remote if this method is used.

The method described by Mr. Wagner of finishing the water-proofing against the girders or concrete is decidedly the most satisfactory of any in the speaker's experience. However, he can see no other object in painting the surface with which the sealing material comes in contact before putting such material in place, than to secure satisfactory cohesion. As there are materials on the market, sold under the name of expansion joint cements, which cohere perfectly without a primer, it would seem cheaper and safer to use them.

As to methods of protecting a water-proofing course on a solid-floor bridge, the speaker knows of no more satisfactory method than brick laid in cement grout, being careful to fill all joints in the brick by pouring mortar over its surface after it has been laid, and sweeping it thoroughly into all joints. It does little good to fill the joints of the brick with asphalt, as even the best hard burnt bricks are porous and offer little obstruction to the passage of water. The use of an asphalt mastic for a protecting coat is an expensive luxury, and by no means a necessity. The argument, at best, for indulging in this luxury is that it is an additional safeguard against the failure of the water-proofing. If the water-proofing is efficient, there is no need of this additional precaution, and it is useless to put in any water-proofing material unless it is good.

One more point is that we are living in an age of intense specialization, and it is possible, to-day, to obtain specialized service without paying a premium. Therefore, the method of letting a water-proofing contract, under which a water-proofing manufacturer is required to do all the concrete work, as well as applying the water-proofing and putting in place the protection coat, does not work out to the best advantage of the railroad. The water-proofer can lay the water-proofing material and place the protection. That is his business, in which he has specialized. He is not a specialist in concreting. The railroad would get better and cheaper concrete, either by letting that work to a contractor whose business is concrete, or by doing it itself. This plan has the disadvantage of being a little more troublesome, and, to a certain extent, of decentralizing responsibility. On every piece

of construction work, however, railroads always maintain an inspector to see that the specifications are carried out, so that these objections are of little moment. As far as is known, it is the general custom on all but four of the Eastern roads, to let a general contract for all the work to a concrete man and to require him to have the water-proofing done by experienced water-proofers with a material approved by the chief engineer. Of these four, two do all their own concreting and water-proofing with their own crew, purchasing the material they prefer to use. The other two use the method advocated by Mr. Wagner. Mr. Gardiner.

As to water-proofing specifications generally, it does not seem as though we had progressed as rapidly in their formulation as in specifications for other forms of structural work. Our engineers are thoroughly aware of conditions which must be met. They are familiar, too, with the general processes of manufacture of the various materials which are designed to meet these conditions. It would not then seem a difficult task to draw a standard specification which would demand the qualities which a water-proofing course should have, and require the manufacturers to produce such a product. Such a move would spur the manufacturer toward improvement, standardize the water-proofing industry, and give the railroads improved service and ever-improving materials.

HENRY H. QUIMBY,* M. AM. SOC. C. E.—Has the author ever tried coal-tar pitch for water-proofing? If not, why not? Also, why is asphalt mastic generally regarded as an undesirable material, as stated in the paper? Also, why does the author call the drainage of the bridge seat a bad detail, when the only examples of it which he mentions have given no trouble? Generally, the bridge seat is exposed more or less to rain, and, therefore, water will reach it independently of any drainage from the bridge floor; consequently, it is good practice to drain the bridge seat back from the face, even when the floor does not discharge on it. This has been the speaker's practice, and it has proved satisfactory. It ought to be possible to secure such drainage anywhere without soiling the face of the abutment, and only a direct connection of the floor drains to a sewer, which is not always available, will be less subject to freezing and overflows in cold weather. Mr. Quimby.

One of the written discussions seems to advocate the use of cement mortar for sealing against the webs and girders. That indicates a great deal of faith in the adhesiveness of cement mortar to steel. Observation of a number of cases shows that the tendency of cement concrete is to leave the surface of the web of a girder, no matter how elaborately it has been made fast to it by embedded steel anchors. This is probably due quite as much to the expansion and contraction of the surface from moisture as it is to vibration, for it has ap-

* Philadelphia, Pa.

Mr. Quimby. peared on structures which are subject to very little vibration. The surface of the concrete, absorbing water, expands against the steel and is probably compressed—permanently compressed—and when it dries out, it pulls away from the surface of the steel, leaving an opening which admits water. An adhesive and plastic substance, like that described in the paper, in a groove from which it cannot flow, seems to be necessary for water-tightness. A sloping hood over this is in the nature of counter flashing, and to the extent that it sheds water from the joint below it, it will certainly reduce the danger of leakage.

The suspended gutter under the bridge floor has its objections—it becomes completely dammed up with dirt, and, unless made very stout, will be dented and bent up. Also, generally, it must be either small and of flat grade or it compels a reduction of the floor depth by trenching on the clearance. Therefore, the most satisfactory design should be the shedding of the floor drainage over the back wall, and also draining the bridge seat through the back wall. Ample capacity for vertical drainage back of the abutment can be afforded by large pipes either to a porous stratum of gravel, if one is there, or to a sewer if one is available, or, failing both these, through the sidewalk to the street gutter. All these can be made accessible for cleaning out obstructions, and where a direct connection to a sewer is not available any drain is likely to freeze up. In freezing weather, the type having a flow over the back wall will be least interfered with.

Mr. Lawrence. RALPH J. LAWRENCE,* ASSOC. M. AM. SOC. C. E. (by letter).—Although the object of this paper is to submit a specification for the design, materials, and manner of application of the water-proofing of railroad bridges with solid steel floors, and to call attention to the necessary details that should be considered by the designer when engaged on such structures, it would be well to know, in view of the author's experience with work of this class, just what method or kind of water-proofing he would suggest for the different types of bridges, water-tightness, durability, and cost being considered.

The writer wishes to refer particularly to the latter part of the Synopsis, where the use of felt, fabric, or asphalt mastic is discussed. As the author does not mention coal-tar pitch, he is evidently of the opinion that the floor of a steel railroad bridge is no place for such material, and, if such is the case, the writer agrees with him.

As to the use of felt, fabric, or asphalt mastic, the writer desires to know which of these materials the author thinks the proper one to use in designing a water-proofed structure, all conditions being equal.

The paragraph referred to states that, if hard pressed for room, asphalt mastic is indicated, thus leaving one under the impression

* Philadelphia, Pa.

that felt or fabric is more satisfactory than the mastic, and that this material should only be used where necessity demands it. Mr. Lawrence.

The felt or fabric, with the brick protection, takes up approximately 2 in. more room than the mastic, and, if they are considered superior to the latter, it would appear to the writer that this slight difference in thickness should not compel its use. The concrete filling required on most bridges must be given a superficial grade on account of drainage requirements, and it would be better to do with a little less ballast at the summits than use an inferior water-proofing material.

It is admitted that felts and fabrics are subject to disintegration, and that a properly prepared mastic is more durable than either of them; then why not use mastic, not only where there is barely enough room to get it in, but where there is sufficient clearance to apply the details used in water-proofing with felt, as shown in Figs. 4 to 9, inclusive.

It is believed that a properly prepared asphalt mastic, $1\frac{1}{2}$ in. thick, is more water-proof and durable than a few films of asphalt placed between several layers of felt or fabric, provided cracks, caused by contraction and the deflection of the steelwork, can be avoided.

Although quite a number of bridges designed by the writer under the author's supervision are water-proofed with felt, and are tight and giving satisfaction, he is of the opinion that a mastic properly prepared with low-melting-point asphalts, properly placed and protected, will prove more effective, more durable, and cost less than the felts or fabrics. The mastic should be placed on the concrete base, and not directly on the steel. It should be laid in blocks, 1 or 2 in. apart, the joints being placed at stiffeners, gussets, at the ends of column-bearing girders, etc., and filled with a ductile, adhesive asphalt, having a low melting point such as that specified for the web and stiffener joints. The mastic should then be protected with about 2 in. of granolithic concrete, reinforced with welded wire cloth, to exclude dirt and retain the asphalt during hot weather.

It is hoped that the author will supply additional information on the subject he has so ably presented for consideration.

A. T. GOLDBECK,* ASSOC. M. AM. SOC. C. E. (by letter).—As has been indicated by Mr. Wagner, one of the very important properties to be possessed by a water-proofing material is that of adhesiveness, and this is of special importance in materials for railroad bridges, as the excessive vibration in these structures tends to loosen the adhesive bond. It is realized that different kinds of water-proofing compounds vary in this property of adhesiveness, depending on their characteristics and, obviously, on the surface to be treated. Mr. Goldbeck.

For the purpose of obtaining quantitative information on the relative degree of adhesion of various kinds of water-proofing materials

* Philadelphia, Pa.

Mr. Goldbeck.

to the surfaces of different characters, the writer has conducted a series of tests, the results of which may be of interest at this time. These tests were undertaken in connection with the water-proofing studies of the author and Mr. James W. Phillips. Before the adhesion of bituminous materials could be obtained, it was necessary to develop an adhesion test, and the scheme finally adopted, although not entirely satisfactory, seemed most nearly to conform to practical conditions. The method of testing the adhesion was as follows:

Specimens, whether of steel or concrete, were prepared, 3 in. in width. A strip of cotton duck (Ottawa sand bagging), 3 in. wide, was immersed in the bitumen to be tested when liquefied at a temperature of about 300° Fahr. The treated duck strip was laid on the steel or concrete specimen, while still hot, and rolled with a steel roller under moderate pressure until cool. The concrete specimens were mixed in the proportion of 1:2:4, and were about 1 week old when the tests were made. In each case the cut-back treatment was applied at least a few hours before the bitumen was to be tested, and with the concrete surfaces thoroughly dry and clean. One day after the preparation of the specimens, they were placed in the tank of a ductility machine containing water at 40° Fahr. After an immersion of about 1 hour, the treated duck strip was peeled off at the rate of 5 cm. per min., and the load was read on a spring balance attached to the moving carriage and to one end of the strip. The spring balance was parallel to the specimen, and, as the carriage was moved along, the duck strip was peeled gradually (not slid) from the surface of the specimen. In general, the load recorded by the spring balance remained fairly uniform throughout each particular test, and the average load was taken as representing the adhesion. The results of the test are given in Table 4.

These results are very instructive, and point out quantitatively the adhesive qualities of different types of materials to various kinds of surfaces. Note how much better the adhesion is with steel than with concrete. With few exceptions, this seems to be a general rule, and shows that where the bitumen comes in contact with two surfaces, one of steel and the other of concrete, particular care must be taken to obtain good adhesion to the concrete. Such a condition arises in the pocket of bitumen, called for in Paragraph 41 of Mr. Wagner's specifications, adjacent to the steel girders. Note the almost entire lack of adhesion of the bitumen to the untreated surface of concrete, shown in Column 10 of Table 4. The failure here was really a concrete failure, due to the stripping off of the non-cementitious layer of laitance from the troweled surface. A cut-back treatment applied to both steel and concrete in general aids the adhesion, especially in the case of the concrete. In Column 13 of Table 4 (concrete specimens) are given the results obtained with a surface freed from the top

Mr. Goldbeck.

TABLE 4.—ADHESION TESTS OF WATER-PROOFING BITUMENS.

	LOAD REQUIRED TO STRIP OFF DUCK STRIP, 3 IN. WIDE, TEMPERATURE, 40° FAHR.																DUCTILITY, IN CENTIMETERS, AT:			PENETRATION.			ANALYSES.				
	Adhesion to Steel Plates, in Pounds.								Adhesion to concrete, in Pounds.																		
	Mill scale not removed.				Scraped and wire-brushed.				Troweled surface.				Surface next to form.														
Compounds.	Untreated.	Painted with cut-back.	Painted with red lead, then cut-back.	Painted with red lead.	Untreated.	Painted with cut-back.	Painted with red lead, then cut-back.	Painted with red lead.	Untreated.	Painted with cut-back.	Acid treatment.	Acid treatment plus cut-back.	Untreated.	Painted with cut-back.	Acid treatment.	Acid treatment plus cut-back.	40° Fahr.	77° Fahr.	110° Fahr.	40° Fahr.-200g-1 min.	77° Fahr.-100g-5 sec.	At 77° after 5 hours heating at 325° Fahr.	Loss on heating for 5 hours at 325° Fahr.	Flow point.	Melting point.	Soluble in CS_2.	Percentage of total bitumen soluble in 88° P. E.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)
A.	16	24	32	24–34	18	24	28	26	0	1¾	½	3½	¼	2¾	1¾	7¾	4¾	11½	32	34	47	29	1.28	140°	167°	99.94	69.63
B.	14	18	20	18	8	12	14	9	0	¾	0	2½	½	3½	1¾	6	2½	3¼	4¼	54	54	38	1.70	161°	197°	99.96	69.26
C.	24	32	34	34	24	48	50	34	0	1¾	¾	20	15	30	20	30	6¾	62	85	26	44	28	0.90	125°	161°	99.90	70.54
D.	10	20	24	17	16	20	20	16	0	1	0	4½	0	9	1	11	1¼	3¾	5½	21	24	21	1.79	203°	254°	99.93	59.68
E.	30	34	38	32	26	30	32	20	0	1½	¼	3½	3	12	3	14	3½	11	16½	31	34	19	2.06	149°	179°	77.75	71.27
G.	36	46	50+	35	28	44	50	44	1	5½	1	3	4	35	20	24	6¼	70	74	34	70	34	1.34	122°	153°	99.95	72.22
H.	22	26	30	28	20	22	24	20	⅛	3	1¼	5	¼	4	¼	8	3¼	4¾	7	28	38	22	4.10	153°	199°	99.94	61.78
J.	26	30	28	28	26	28	32	32	¼	1½	½	5½	½	12	0	10	3½	6½	11	25	31	16	3.69	160°	192°	99.95	62.55
L.	24	20	24	20	24	18	20	16	0	1½	1¼	4	3	16	5	12	4½	10	25½	46	63	35	0.85	149°	170°	99.00	71.63
M.	24	20	24	24	22	24	20	20	0	1¼	½	6	¾	16	2	14	1½	7	16	18	27	18	1.25	154°	190°	99.90	59.76

Mr. Goldbeck. layer of laitance by using dilute hydrochloric acid, and then painted with the bitumen under test cut back with gasoline. Note the large increase in adhesion to the concrete resulting from this treatment. These results should be of interest to the builder of concrete roads to which a bituminous surface treatment is to be applied. They indicate the absolute necessity of ridding the surface of the non-cementitious layer on the concrete before the application of the bitumen. In Columns 14 to 17, inclusive, are given the results obtained on the concrete surface adjacent to the wooden form. They are generally higher than those obtained with a troweled surface.

Referring to the results of adhesion to the steel surfaces, it will be noted that there seems to be relatively good adhesion irrespective of the character of the surface, whether scraped, unscraped, or painted with red lead. A cut-back treatment, however, seems to be quite generally beneficial. On the steel plates used as specimens, there was very little loose mill scale and no rust. It is well to rid the surface of all loose scale and rust before applying the bitumen.

Regarding the ordinary physical characteristics of the bitumens used, correlated with their adhesive properties, it would seem that ductility and adhesion go more or less hand in hand. Thus, compounds having high ductility seem to have good adhesive properties. It seems well, therefore, to specify a fairly high ductility in order to insure more nearly a material of good adhesion. The value, 20 cm., specified by Mr. Wagner, would seem to insure a material that will adhere well to both steel and concrete surfaces when these surfaces are properly treated. Perhaps it would be well to specify a definite ductility at 40° Fahr., as this temperature approaches more nearly the dangerous one at which the adhesive bond is likely to be broken. As the hardness of the material likewise influences the adhesion, it would be advisable to specify definite penetration limits.

Mr. Auryansen. F. Auryansen,* M. Am. Soc. C. E.—About ten years ago, when extensive grade-crossing elimination on Long Island was under consideration, among the first problems to be solved was the kind of bridge floor. Although much more expensive, the solid floor was adopted, because, though it would allow continuous ballasted tracks, it would also prevent grease, ashes, coal, or other materials from falling to the street below. Of the three kinds of solid floor—timber, steel, or a combination of reinforced concrete and steel—the latter was adopted as promising to be the most permanent and satisfactory, provided it could be protected from water.

One of the most important reasons for water-proofing bridges is the reduction of danger from electrolysis. In the future, as in the past, the majority of water-proofed bridge floors will unquestionably be within

* Jamaica, N. Y.

or near towns and cities, where stray currents from electric light, power, and traction systems are to be expected. Furthermore, electric traction for suburban service is in so much favor in large centers of population that its extension to through business is, in a number of cases, only a question of time; and although, with the further development of electric utilities, adequate means may be devised to reduce or eliminate stray currents, in the present state of the art, the floors of bridges now built should be protected from this insidious danger. Mr. Auryansen.

Wet concrete is an electrolyte, and, at the other extreme, dry concrete is a poor conductor; hence the reinforced concrete floors, not only of railroad, but of highway, bridges, should be kept dry, that is, water-proofed; and it should be the aim in all designs to simplify the problem of water-proofing, so that the desired end may be reached, namely, a tight floor.

Expedients to secure this result are, in order of desirability:

1.—Such grades that the water will run off quickly;

2.—Drainage at back walls;

3.—Water-proofing, either by the integral, mastic, or membrane method;

4.—As a last and unavoidable resort, drain pipes.

Good Grades.—A top of floor grade of 0.7% parallel to the tracks is the minimum slope used. When the track grade is less than this, a summit is provided, and the drainage is carried thence to each end of the span on 0.7% grades.

Drainage at Back Walls.—For several years, it has been the practice on the speaker's lines to place a course of loose rubble against the back of the abutments for the full width of the bridge floor, and from the original ground level up to the floor, effectually disposing of the wash from the bridge and eliminating the unsightly leakage often found along the horizontal joint between the slab and abutment.

If the water can be simply conducted through the floor and allowed to fall freely to the ground, as in the case of a viaduct on a private right of way, no serious objection can be urged, although in cold weather there may be danger to trackmen or others beneath from falling icicles. Gutters over streets, however, are a nuisance; birds' nests clog them, and high loads on passing vehicles injure and are damaged by them. Pipes, leading from the bridge floors to the street or sewer, freeze and burst, and in the vicinity of New York, for several years, they have been avoided.

About 1909, a few through, girder bridges were built with no deck water-proofing, although flashing angles were provided on the web. Care was taken in the concrete work to secure a dense mixture, and the surface was carefully troweled. Although there is some leakage,

Mr. Auryansen. no rust has appeared; but no more floors are built without water-proofing.

No faith is placed in the "integral" method of water-proofing, as it cannot cope with slab cracks, construction joints, or the shrinkage of the concrete from the steel. The vibration caused by passing trains also inevitably shakes or jars the concrete while setting, thus inducing very narrow cracks, especially along the under side of horizontal or inclined surfaces of the steel, and at such cracks, capillary action draws in enough water to cause continued dripping below for days after a storm.

The mastic method is fully treated by the author. The only example of this type with which the speaker has had experience is for a very shallow trough floor, 11 in. from base of rail to clearance, built in 1903. The ties rested on stirrups in the troughs, and the space around them was filled with a soft asphalt mastic. Rail creeping has forced a little of the mastic out, but the remainder is still in good condition.

The membrane method is the principal one used on the Long Island bridges. Five-ply felt served with coal-tar, with the middle layer reinforced, has been used most extensively, and with very good results; burlap with asphalt pitch was used on one bridge; and a number have been protected with a single-ply cotton fabric dipped in special pitch, and after being laid on the freshly coated concrete, the laps and edges are sealed with a hot iron. Recently, a few floors have been treated with a two-ply cotton fabric impregnated with asphalt pitch. The five-ply tar membrane is very satisfactory on level or slight grades, but on inclined surfaces, the tar has a tendency to settle away from the top edge of the membrane.

On deck structures, with flat surfaces and no corners or projections, the conditions are ideal for good workmanship. Perhaps the most difficult places to make tight are where the edges are flashed up on parapets. Until within a year or so, a 3-in. lip or coping was cast in the parapet wall and the membrane was carried up and sealed under the coping. A later modification is the insertion of a copper flashing strip in the wet coping concrete, the projecting portion being bent down over the edge of the finished membrane. On another recent job the membrane, shown by the dotted line in Fig. 10, was carried over the top of the unfinished parapet, on which the coping was then built. In either case, it is important to follow promptly with the protection course of brick or concrete, in order to prevent the sagging of the membrane and the unsealing of the edge.

It is more troublesome and expensive to make through bridges tight, but it is again mainly a question of sealing the edges of the membrane properly. Engineers seeking tight floors must disagree emphatically with the fabricator's suggestion that the flashing angles be discarded, although, at the same time, sympathizing with him as

Mr. Auryansen.

to their cost. Angles of ¼-in. metal are too thin and will deform too easily; ⅜-in. should be the minimum. The intricate fitting at gussets can be eliminated to a great extent by making the longitudinal-web flashing angle continuous between web splices; the gussets may be cut to permit this, and the small flashing pieces may be attached thereto at

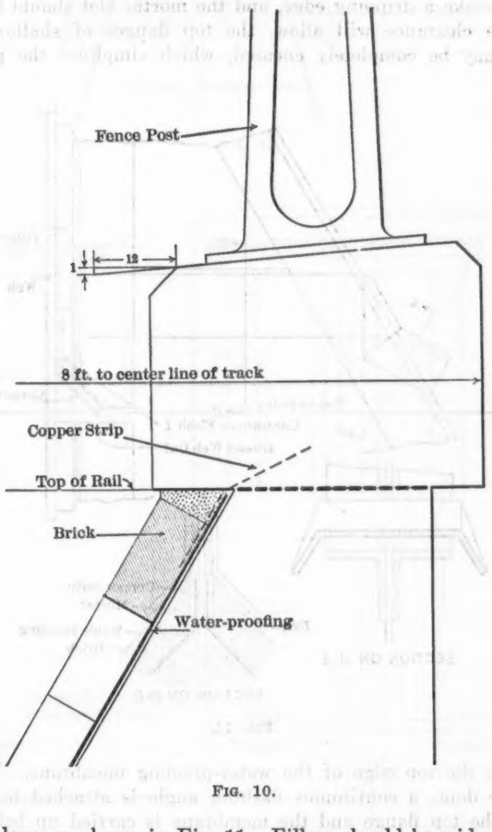


FIG. 10.

a higher plane, as shown in Fig. 11. Fillers should be wider than the flash-angle leg and project above it. The edge of the gusset may be protected by small flaring steel castings or by a cover-plate bent outward at the bottom to flash the membrane and brick. Reasonable care at the shop in fitting these details will make it easily possible

Mr. Auryansen.

for the painters in the field to caulk completely the few small cracks, either with paint skins or red lead putty. A bridge now at the shop is to be fitted with copper strips under the flash angles; these strips are not punched for the rivets, but simply inserted and held in place by the grip of the latter. On completing the membrane and its protection, a trowel-cut should be made in the mortar under the flash-angle to make a dripping edge, and the mortar slot should be painted.

Where clearance will allow, the top flanges of shallow, through girders may be completely encased, which simplifies the problem of

FIG. 11.

protecting the top edge of the water-proofing membrane. When this cannot be done, a continuous flashing angle is attached to the lower edge of the top flange and the membrane is carried up behind it, as shown in Fig. 12.

Other troublesome features are the provisions for contraflexure and expansion. Where the former is anticipated, either the slab and membrane should have additional reinforcement, or a flexible joint should be provided. If possible, expansion should be allowed only at

Mr. Auryansen.

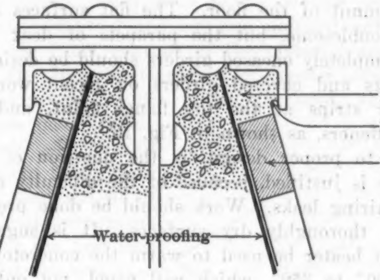


FIG. 12.

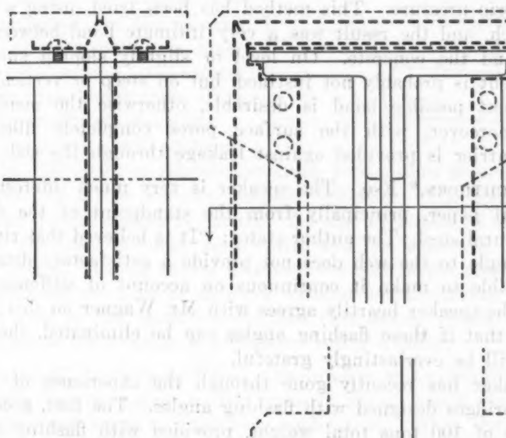


FIG. 13.

Mr. Auryansen.

the crown or summit of the floor. The flat surfaces at such points are not very troublesome, but the parapets of deck slabs and the upper part of completely encased girders should be designed carefully. Concrete parapets and covered girders on recent work have **U**- or **V**-shaped copper strips at the top flange joint, and vertically at abutting end stiffeners, as shown in Fig. 13.

Second only to proper design, is the question of workmanship. The utmost care is justified, because of the difficulty and expense of finding and repairing leaks. Work should be done preferably in hot weather and on thoroughly dry surfaces. It is suggested that an asphalt pavement heater be used to warm the concrete to a temperature of from 300° to 350°, which will expel, not only any surface moisture, but much of the air in the pores of the concrete; pitch applied to this hot surface will be kept liquid until forced into the pores by atmospheric pressure. This method has been tried, using a large gasoline torch, and the result was a very intimate bond between the membrane and the concrete. On level or slightly sloping surfaces, this refinement is probably not justified, but on steep or vertical surfaces, the best possible bond is desirable, otherwise the membrane will sag; moreover, with the surface pores completely filled, an additional barrier is provided against leakage through the slab.

Mr. Fichthorn.

J. H. FICHTHORN,* ESQ.—The speaker is very much interested in this excellent paper, principally from the standpoint of the design of the structural steel. The author states: "It is believed that riveting a flashing angle to the web does not provide a satisfactory detail, as it is impossible to make it continuous on account of stiffeners and gussets". The speaker heartily agrees with Mr. Wagner on this point, and is sure that if these flashing angles can be eliminated, the steel fabricator will be everlastingly grateful.

The speaker has recently gone through the experience of fabricating two bridges designed with flashing angles. The first, a double-track bridge of 100 tons total weight, provided with flashing angles ¼ in. thick, which, including the fittings around the brackets, weighed 900 lb. The detailed drawings for this 900 lb. of flashing cost $128, and the fabrication cost $300, making a total of $428 for 900 lb. of material, or $4.28 per ton, on the total weight of the bridge, due to flashing alone. Special care was taken to make a good fit, but the results showed that it was practically impossible to make a tight job around the brackets.

On the second bridge, a five-track structure of 300 tons total weight, a new method was adopted, for the purpose of obtaining a better fit and reducing the cost. No detailed drawings were made. The work came from the shop completed, except for the flashing. A draftsman

* Chf. Engr., Lewis F. Shoemaker & Co., Pottstown, Pa.

Mr. Fichthorn.

took cardboard (which material is nearly always used for templets of small details), and fitted and cut and notched until the resulting templet showed the exact contour of the material around which the flashing was to fit. In order to permit of greater accuracy in cutting and bending the material, $\frac{1}{8}$-in. metal was used, and, as before, a special effort was made to obtain a tight job. The result was a cost of about $600, or $2 per ton on the total weight of the bridge, without very much improvement over the first case as far as a tight job was concerned.

The use of malleable-iron fittings around the brackets has been suggested, but the speaker has found that, owing to inaccuracies in casting, the results are very little better than, if as good as, steel-plate flashing.

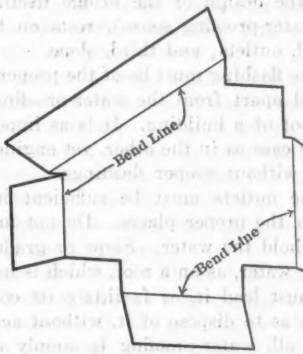


FIG. 14.

An illustration will probably show the difficulties of fabrication much better than description. Fig. 14 shows one of the simpler templets involved. The reentrant cuts, slits, and bends should be noted. As the thickness of metal is increased, the work becomes more costly and less accurate.

The use of flashing angles increases the cost of the steel very much and lengthens the time of delivery without accomplishing the desired results, and, in the speaker's opinion, this money might well be expended on water-proofing materials of better quality.

Mr. De Knight.

EDWARD W. DE KNIGHT,* ESQ.—Failures in water-proofing on bridge floors are due primarily to two causes: First, wrong design of the structure; and second, imperfect application, rather than the kind of materials used.

Imperfect application usually results from two causes: First, the use of labor not skilled in that special class of work, and second, lack of sufficient time and facilities by the railroad for the proper execution of the work.

In the past fifteen years or more, the speaker has examined several thousand bridges in the United States, as well as abroad, and in not one in ten has the floor itself been found to be properly graded. How, otherwise, can the water be drained off? The speaker has never been able to understand why a bridge designer considers in such a peculiar, abstract, almost supernatural, light, a bridge floor to be water-proofed.

* President, The Hydrex Felt and Engineering Company, New York City.

Mr. De Knight. It is nothing more or less than a roof. Yet a bridge engineer will invariably use, on a bridge floor or deck, a design and materials which he would not dare to use on the roof of a building.

A roof has flashing, it has outlets, and it has a slope. Any consideration, therefore, of water-proof bridge floors must begin, first, with the design of the bridge itself. This design, to be correct (in the water-proofing sense), rests on three essentials: First, flashing; second, outlets; and third, slope.

The flashing must be of the proper kind, and it must be independent of and apart from the water-proofing, precisely as is the flashing on the roof of a building. It is as impossible to do without this flashing in one case as in the other, yet engineers attempt to water-proof bridge floors without proper flashings.

The outlets must be sufficient in number, correctly constructed, and in the proper places. Do not make the bridge floor dead flat, so as to hold the water. Slope or grade the floor so as to shed or drain off the water, as on a roof, which is never flat. One cannot fight water; one must lead it, or facilitate its continuous flow to drains and outlets so as to dispose of it, without accumulation, as rapidly as it falls. After all, water-proofing is simply a part of the general scheme of drainage.

As to materials, something has been said about mastic. One would not place on the roof of a building a mastic, like an asphalt pavement, in order to make it water-tight, because it is known that a mastic or asphalt pavement will crack in time and let the water through the cracks to ruin the interior of the building. Therefore, one is afraid to use or depend on mastic. Yet an engineer would use mastic on an arch or a bridge floor to make it perhaps water-tight, when he well knows that it is not dependable for the roof of a building and that it is neither good practice nor roofing practice to use it thus. The speaker repeats, that he cannot understand why a bridge designer considers so peculiarly and abstractly an arch or bridge floor to be water-proofed, as compared to an ordinary, every-day roof.

One word as to the protection of the water-proofing. The speaker's experience is that hard-burned, vitrified brick, laid and grouted in cement mortar, is the best protection for the water-proofing membrane, and is superior to mastic: First, because the brick can be readily removed in sections, and the water-proofing examined and repaired, if necessary; whereas mastic cannot be removed so readily. Second, mastic also has the serious disadvantage that at times the ballast and ties will sink into it, especially in hot weather, and form pockets to hold water and thus hasten decay. This has proved to be so repeatedly. If the mastic is made soft enough not to crack in winter, it will be too soft in summer, and if made hard enough not to soften too much in summer, it will crack in winter. A bridge floor exposed to extremes

of weather and subject to heavy traffic, presents conditions quite different from the floor of a freightshed or warehouse. It was because of the failure of mastic years ago, on arches, viaducts, and similar work, that several roads, as, for instance, the Pennsylvania Railroad, discontinued its use, especially on arches and bridge floors, and adopted the more yielding and dependable membrane method. The use of mastic, therefore, is retrogressive and not progressive. Mr. De Knight

Under materials, Mr. Wagner refers to felt, burlap, cloth, and asbestos, and mentions a trial of one or two layers of cloth—possibly two—on a bridge floor. Now, it is a mistake to use on a bridge floor, subject to the impact of heavy trains and to almost constant vibration, fewer than four layers of any material, no matter whether it be cloth, felt, or burlap; and it would be far better to use five or even six layers. Five is the general standard. The speaker assumes that cloth is specified in only two layers because of its great expense.

The speaker discusses all these materials without bias because his Company manufactures treated cloth, treated felt, treated burlap, etc., and, from a manufacturer's standpoint, it does not matter whether one or the other is sold. From a technical, engineering standpoint, however, and from a long and varied practical experience in the sale, use, and application of the materials, he can recommend only those materials which he candidly believes are best suited for the special purposes and conditions. He would not for a moment, therefore, recommend the use, for water-proofing bridge floors, of only two layers of any one of the previously mentioned materials, whether felt, cloth, or burlap. Two layers of cloth, for example, in place, cost as much as five layers of a good, tough, wool-mixed felt. These five layers form a membrane or stratum of water-proofing approximately $\frac{3}{4}$ in. in thickness, whereas two layers of cloth, or felt, or burlap, form a stratum about only $\frac{1}{8}$ in. thick. It requires no argument to prove that, all things being equal, a stratum of water-proofing about $\frac{3}{4}$ in. thick will be more durable than one only $\frac{1}{8}$ in. in thickness. If the stratum is not durable, of what value is it? It is the material in a mass which insures its durability. The additional strength in the thin stratum, because it contains cloth, does not add one iota to this durability, because strength cannot make up for mass and durability. Therefore, if one wishes to use cloth, or burlap, or felt, to obtain the durability which is insured by the mass, one must use not fewer than four layers, or better, as previously suggested, five or six layers. However, as four and five layers of the right kind of felt have given perfect service for all practical purposes for many years, yielding in the mass to all the natural contraction, expansion, and settlement of concrete, and vibration on bridge floors, it is unnecessary to go to the expense of using four or five layers of cloth costing more than double that of five layers of felt, because this

Mr. De Knight. means an enormous increase in cost for a strength in the cloth, which is necessary only in imagination and not in fact. The factor of safety, in this case, does not require 100% increase in cost. Strength in water-proofing will not hold concrete together or prevent it from cracking. Abundant experience and practice show that four or five layers of a good felt, forming a water-proof mass approximately $\frac{3}{8}$ in. in thickness, in practically every respect and everywhere, has served satisfactorily the mobility of concrete, and at half the cost of cloth, the strength of which is not necessary. There is no real need, therefore, for incurring double expense for an unnecessary strength.

Recently the speaker inquired of an engineer who had designed a large power-house, "What do you purpose specifying for the foundation water-proofing"? He mentioned two layers of fabric. The speaker inquired what he intended to use on the roof. He answered, following the usual custom: "Five layers of felt". The speaker asked him if he had not the principle reversed. Why not invert the structure, and place the five layers below grade and the two layers on the roof? When asked if he would specify two layers of any felt or fabric for a roof, he said: "No, because I do not think it would endure". The speaker replied: "If two layers would not serve on the roof, where you have simply the elements to contend with, and you can always reach and inspect the material, by what logic do you assume that two layers will endure below grade, where it will always be subject to water, the salts in the earth, alkali in the concrete, etc., and where you can never see or get at it? This being so, would it not be better to use, below grade, twice instead of one-half the quantity of materials, that is, if five on the roof, why not ten below grade? This would seem the logical procedure". He remarked that this phase of the problem of water-proofing had never previously occurred to him; that if the mind of an engineer could be brought from other things to think seriously of the technical side of water-proofing in such a light, it would do an immense amount of good.

In considering whether to use felt, burlap, cloth, or asbestos, we again revert to the question, not of strength, but durability. This should be made as clear as possible. Durability in all these materials is based, therefore, on their composition, and on their fibers, which may be analyzed as vegetable, animal, or mineral.

Burlap, for instance, is of a vegetable fiber (jute), the least durable of all the fibers, the most difficult, and one almost impossible to water-proof properly. It misleads architects and engineers into its use, because it seems so plausible to talk about its strength. This idea of strength, as the speaker has attempted already to explain, is a fallacy. Reinforcing concrete is based on an entirely different principle from attempting to reinforce asphalt with burlap. As already

Mr. De Knight.

explained, the essential thing in water-proofing is not strength, but to make it water-tight and durable. Burlap decays sooner than any of the fibers. Why use it, therefore, simply because of its temporary strength, *i. e.*, during application? Durability is thus being sacrificed for utility.

Cloth is also of a vegetable fiber (cotton), and a weak and also very perishable fiber. As a fiber *per se*, it can be easily water-proofed, but when twisted into a thread and woven into a cloth, the fabric shrinks, and it is difficult to get the water-proofing preservative, not only into the twisted threads, but also into the interstices. To accomplish this, the cloth is run through the saturating compound slowly. It is thus apt to be burned, and in this way there is destroyed at the beginning, not only its strength, but its durability and its real intrinsic worth.

The best material for water-proofing is a good felt composed of cotton and wool fibers—not the twisted, but the natural fiber. A good felt should contain from 25 to 30% of wool fiber. We know very well that wool is much more durable than cotton, and, therefore, that a mixture of wool and cotton produces a more lasting material than one which is composed entirely of cotton or vegetable fiber—such as cotton cloth.

Wool fiber or hair (animal fiber) is of a horn-like structure and contains oils which preserve it indefinitely.

With reference to asbestos paper: the speaker desires to be very frank in saying that he wishes to do all he can to warn against its use for water-proofing purposes. The following facts will be of interest and value to any one contemplating the use of this material for water-proofing, and should have earnest consideration.

To begin with, the fiber (so-called) in asbestos paper, is not a fiber. It is a splinter or sliver, a sloughing off lengthwise of the crude asbestos, which is the common mineral, hornblende. Hornblende contains silicate, magnesia, and lime or iron. The magnesia or lime in the asbestos tends to destroy any oil with which the paper is treated in water-proofing it. This explains why asbestos paper, saturated in asphaltic oil, dries out so soon and becomes dead.

The fiber or sliver itself is not a natural fiber; that is, it does not curl and felt into a natural felt like wool fiber with its many pellicles. Examined under a microscope, we find an asbestos fiber or sliver to be what is termed a straight-out or dead fiber. Therefore, to make it hold together in a paper, it must be artificially cemented together or sized with starch, tapioca, or something similar, making a paper absolutely lacking in flexibility and tensile strength. Dip the paper into water and it disintegrates by its own weight. The longer fiber or sliver of asbestos is so expensive that it is used in only the high-priced twine, rope, and asbestos cloth. It is too costly

Mr. De Knight. to use in asbestos paper, which is composed of the weak and broken ends, or the refuse which cannot be used in twine and cloth, therefore the additional necessity of cementing the ingredients of this paper by sizing.

About fifteen years ago, the speaker was engaged in the manufacture of the first (so far as he knows) asphalt-treated asbestos paper made in the United States for use in below-grade water-proofing. It was first specified (in 1899-1900) by William Barclay Parsons, M. Am. Soc. C. E., then Chief Engineer of the New York Rapid Transit Subway Commission. The Subway was the first great work of its kind in the country. When Mr. Parsons set out to design and build it, there were no established guides. He had to establish new standards. There were no standard specifications for water-proofing—no treatises on the subject. "Modernized Waterproofing" by the speaker, in 1902, was the first of its kind. Asbestos paper treated with asphalt, was specified by Mr. Parsons for the original Rapid Transit Subway work, on the theory that as the mineral fiber or sliver from asbestos was fire-proof, it would be decay-proof, and, therefore, an ideal material for below-grade water-proofing. The theory did not work out in practice, however, because of the inherent defects in the fiber itself, as the speaker has already attempted to explain. This was not discovered until several thousand rolls of asbestos paper had been used in the work. The great trouble was that the material was so friable and weak that it tore easily when the workmen tried to handle it. In working it into corners or into angles formed between the floor and the wall, the fingers of the workmen easily went through the paper. Their shoes also went through it, when they walked on it. It died out rapidly when kept in storage. It could not be pulled, roughly handled, and manipulated like a wool-mixed coated felt. After investigation and the discovery of the real defect, which was inherent in the paper, its use was discontinued, and not a square foot of it was used thereafter.

From the standpoint of the material manufacturer, it matters little whether treated felt, treated cloth, treated burlap, or treated asbestos paper (if desired), is used, but from a technical and engineering standpoint, based on long and varied experience in the several branches —manufacturing as well as constructional—the speaker candidly recommends as the best material a saturated and coated felt containing from 25 to 30% of wool. It can be easily and thoroughly impregnated so as to preserve its every fiber. The felt should be coated, like enameled leather, on its two surfaces. This produces a tough, flexible, absolutely impervious sheet in itself, with no holes or interstices, as in cloth or burlap. The finished material is then as near like skin, hide, or leather—the most natural water-proofing—as it is possible to obtain. The standard method is to use from four to six layers

Mr. De Knight.

of such a coated, impervious, specially-made felt. It is cemented together with a bitumen compound as elastic as the felt. This forms in the mass a thick, tough, water-proof stratum. This stratum is apart from and practically independent of the concrete, which can settle, contract, expand, or crack without injuring this non-conducting stratum or membrane. It was because of the resemblance of such a specially prepared, glazed-coated felt to a skin or membrane that about ten years ago the speaker originated the expression "the membrane method".

Mr. Rhett.

ALBERT H. RHETT,* ASSOC. M. AM. SOC. C. E. (by letter).—Some experiments made by the writer in connection with the construction of the bridges of the New York, Westchester and Boston Railway, may be of interest, because, after three winters' service, the results are entirely satisfactory.

This work involved the water-proofing of 56 bridge structures, including 700 ft. of two-track viaduct over the Mount Vernon, N. Y., water supply, 2 500 ft. of four-track viaduct and the terminal station at 180th Street, New York City, two half-through railroad bridges where the under side of the bridge floor formed the ceiling of the station concourse, 1 mile of four-track subway, with two stations, and many crossings of important city streets and at station entrances. The problem, therefore, seemed to merit special attention, and as previous experience with standard methods did not furnish any great encouragement, it was determined, in sheer desperation, to make a somewhat radical experiment, though this experiment did not involve a departure from the elastic membrane theory, which was believed to be axiomatic in the water-proofing of live-load structures.

Observation had shown that there were two almost universal causes for the failure of a membrane in such structures, namely, the breaking away of the flashing of the membrane from the sides of the girders in half-through structures, due to the deflection of the floor, and the breaking of the membrane over the supports in multiple-span structures due to the deflection of the span.

It was thought that both these difficulties could be corrected if it were possible to devise a membrane, which, instead of being composed of layers of supposedly elastic water-proof fabric, should consist of a compound which would be permanently, and at all temperatures, adhesive, cohesive, elastic, and water-proof. If such a compound could be obtained, it would be self-healing when punctured; would stretch and not break over the supports; and, moreover, a permanently elastic water-proof connection could be made between this membrane and the side-supporting girders by a continuous trench in the concrete base, along the girders at their juncture, filled with this compound.

* Brooklyn, N. Y.

Mr. Rhett. A careful search of the market did not reveal a compound which possessed these qualifications, and it became necessary to resort to the expedient of trying to have one manufactured. After several experiments this was accomplished and a substance was evolved, which, after a year's exposure to the open air, remained adherent, coherent, and elastic at all temperatures, adhering to glass at zero temperature and, though semi-plastic in its normal state, not liquefying except at high temperatures.

This substance, reinforced with burlap, was then used for a membrane, and the side trenches were partly filled with oakum on which the compound was poured. This composition has no destructive chemical effect on the burlap. As used, the membrane consisted of one heavy layer of the compound poured and swabbed on the surface of the floor, on which the burlap was laid and which, in turn, again received a light coat of the compound.

The method also presented a number of practical advantages. It was so easy of application that it could be done with unskilled labor; the compound being non-hardening, could be applied in cold as well as in warm weather; being permanently adhesive, it could be applied to a damp or wet surface; the side trench saved the usual elaborate protective fittings at the juncture of the membrane and the girder web; and, being permanently adhesive and semi-plastic, the cleaning of the surface of the girder, so carefully specified by Mr. Wagner, was entirely immaterial, and no attempt was made to clean this surface, except to brush off the loose dirt. The membrane was covered with a 3-in. layer of concrete reinforced with poultry netting.

This work was done in the winter and early spring of 1912, and in a letter to the writer, under date of January 25th, 1915, the Engineer of Maintenance of the Company, Mr. F. Zogbaum, states that "the water-proofing on all these bridges has proven very efficient".

All the structures, however, were not water-proofed by this method, quite a number of them having been treated with the standard 4-ply felt cemented together with the usual type of compound, and covered with hard-burned brick, run in with the same compound instead of cement mortar, and they have also proved satisfactory up to the present time.

In all cases an attempt was made to reinforce the floor proper, just beneath the membrane, against breaking, which would tend to tear the membrane, and this was done even in the trough floor structures. Longitudinal reinforcement in the form of expanded metal was inserted over all intermediate bents or columns; and over the abutments rods were used which were turned down over the back wall, the floor slab proper being always carried over the back wall, to a ledge thereon made to receive it.

With the exception of the 180th Street Viaduct, no attempt was made to drain the floors except over the back wall, and in few cases was the grade more than 1 per cent. In all cases, however, the water-proofing and its concrete cover, with the poultry netting, was carried over the back wall to the ledge on the back of the abutment. At the expansion end this was separated from the ledge by a layer of paper and compound, and a pad of corrugated asbestos about 2 in. thick was inserted between the face of the back wall and the turned-over portion of the floor slab, to permit of contraction, it being thought that the usual cause of leakage at this point was due to the tearing of the water-proofing by this contraction. Mr. Rhett.

The Hutchinson River Viaduct, which is over the water supply of Mount Vernon, N. Y., is a two-track, solid-floor, deck structure, 700 ft. long, with two expansion joints, and as it was against the law to deliver the drainage on the adjoining property, it was necessary to let it flow the entire length of the structure on the roadbed alone, and though the grade was only 1%, and in spite of the two expansion joints, there has not yet been any complaint or trouble. This structure was water-proofed with the standard 4-ply felt and brick covering, and the expansion joint was made by a U-shaped, sheet-lead roll inserted in the 2-in. opening between the ends of the slabs of the sections, which was filled with oakum and the non-hardening, adhesive, cohesive compound. This opening was protected from the ballast by a steel cover-plate anchored to the slab of the fixed side of the opening.

The three special features of the system, as used, were:

First.—The use of a membrane of which the active water-proofing agent was an elastic compound instead of an elastic fabric.

Second.—The method of attaching the membrane to the face of the vertical girder by a pocket of a permanently adhesive and elastic compound, thus insuring a permanently elastic water-proof joint.

Third.—The special reinforcement, of the surface water-proofed, against cracking that might affect the integrity of the membrane.

As far as three years' service may constitute a test, the results have proven satisfactory.

In any discussion of the subject of water-proofing, it seems inevitable that, sooner or later, three points of controversy will present themselves, namely, felts *vs.* fabrics; non-adhesion *vs.* adhesion; and thickness *vs.* thinness.

Felts vs. Fabrics.—It is unfortunately true that where a bridge structure is submitted to a variation of temperature of from, say, zero to 120° Fahr., its water-proofing membrane is subjected to the

Mr. Rhett.

same disrupting strains at the low as at the high temperature. It is generally conceded to be true, also, that for a membrane to perform its function successfully, it must not only be water-proof, but elastic, and as it is to be subjected to the same strains at low and high temperatures, it must, if elasticity is an essential requisite, be as elastic at the low temperature as at the high one.

Now, in either the felt or fabric membranes, as commonly used, the plying medium is a compound, and although the felt or fabric, as the case may be, is more or less elastic at all temperatures, it is the writer's experience, that the plying compounds are not only very inelastic at temperatures below freezing, but very brittle and friable.

It would seem inevitable, therefore, that at such temperatures the compound will be fractured by the deflection of the structure and impact of the load, and when thus once broken, the disrupted segments can certainly not be re-united until the structure is once more subjected to a temperature equal to that of the melting point of the compound, which temperature is usually specified as much higher than any to which the structure is ever likely to be subjected.

The membrane, then, after having once been subjected to the low temperature will consist thereafter, presumably, of a number of sections of inelastic compound held together by the felt or fabric. As the saturating element of the felt is kindred in nature to the plying compound, the felt will certainly lose elasticity at the lower temperatures, but if its coherent quality is sufficient to withstand, without impairment of its water-proofing integrity, the strain that disrupts the compound, then, of course, it will bridge the break in the compound and bar the passage of water on a return to the higher temperatures. With the fabric membrane, the case would seem much worse, and it is very difficult to comprehend, how, after the compound is once broken, a few strands of cotton or burlap can prevent the passage of water through the membrane.

Non-adhesion vs. Adhesion.—When a membrane is cemented to the surface to be water-proofed by a compound that is rigid and inelastic at low temperatures, it becomes evident that the elasticity of the reinforcement cannot come into play at those temperatures until both that surface and the compound have broken, and that then the effective elasticity is localized to the line of rupture. As far as the integrity of the membrane is concerned, then, it would seem that the membrane should not be cemented to the water-proofed surface, though the possibility of the water getting under it, in case of puncture, when not so cemented is certainly a strong argument in favor of doing so.

Thickness vs. Thinness.—If a stratum of material is truly and inherently water-proof, how can it be made more so by multiplying it? If each stratum is defective, then, on the theory that the defects in successive strata may be staggered by the multiplication of the

strata, there may be some basis for argument. The writer believes that where a material is truly water-proof, any increase in thickness beyond that necessary to cover effectively the surface to be water-proofed, brings with it a very injurious effect in the proportionate decrease in flexibility, and certainly a very undesirable one in the proportionate increase in cost. Mr. Rhett.

A. W. Carpenter,* M. Am. Soc. C. E.—The speaker has been very much interested in this paper, as he has had to design and write specifications for water-proofing for many similar structures. The lines he has followed have been somewhat similar to those set forth by Mr. Wagner. In a good many respects, however, they have been different, and perhaps some of his experiences and differences of practice may be of interest. Mr. Carpenter.

The art of water-proofing solid steel-floor railroad bridges is, as has been intimated, a rather new one, and in the process of development. One of the discussions mentioned the earliest form, at least the earliest known to the speaker, which is the so-called binder that was put in the bottoms of the steel troughs which formed the earliest type of the solid floor.

That binder, although entirely inadequate as a water-proofing material, had the virtue of affording very considerable protection to the steelwork. In a typical case the lower inside surfaces of the troughs were protected only by a deposit of this so-called binder, which was composed of either coal-tar pitch or asphalt, mixed with gravel or sand; or, in some cases, it was thought fit to specify a mixture of coal-tar pitch and asphalt, and then mix that with sand or gravel. A great many of these old troughs, built from 1888 on, which were uncovered many years later, showed that the binder had protected the steel from corrosion very well.

The more elaborate form of water-proofing, which is the subject of this discussion, appears to have been introduced about the beginning of this century, and has been, of course, largely developed in connection with the extension of grade-crossing elimination work in cities.

Mr. Wagner mentions ballast as being one of the main reasons for requiring water-proofing. The speaker thinks that is so, and that one of the ways to get rid of the necessity for water-proofing is to get rid of the ballast. He has experimented to some extent on solid floors with the ties or timber rail blocks embedded in concrete in place of ballast. That does away with the necessity for elaborate water-proofing. A smooth surface of concrete properly sloped carries the water off rapidly, so that it can be taken care of by gutters.

Mr. Wagner thinks it inadvisable to attempt to drain the water over the bridge ends and back walls. The speaker has always thought

* New York, City.

Mr. Carpenter. this the preferable way, where the bridge is on a sufficient grade to obtain such drainage inexpensively, as it generally dispenses with the necessity for the expensive and frequently troublesome drainage system of outlets, gutters, and down-spouts. A grade of 0.7 of 1% on short bridges has been considered sufficient for this purpose.

With regard to carrying water-proofing and concrete over the top of low half-through girders between tracks, the speaker would not advocate this if any increase of track spacing in order to maintain proper clearance should be caused thereby, thinking it would not be worth the cost.

Mr. Wagner objects to girder flashing angles. The speaker has found that form of construction very satisfactory. Instead of stopping at the stiffeners, he has carried the steel flashing around the stiffeners and knee-braces. He realizes that the bridge shop does not like it very well. It involves a great deal of difficult detail, but it seems to give the desired result, so far as the water-proofing is concerned.

Mr. Wagner refers to only one water-proofing material, that is, asphalt. The railroad with which the speaker is connected has used both asphalt and coal-tar pitch, and has had good results with each. Coal-tar pitch is cheaper, having the advantage in that respect. With regard to durability, as far as the speaker's knowledge goes, coal-tar pitch is very durable. He is not so sure about asphalt.

Some fourteen bridges, carrying railroad tracks over streets in Buffalo, and designed under the speaker's direction, are water-proofed variously with coal-tar pitch and with asphaltic compounds, largely according to the methods described by Mr. Wagner. They have been in service from 4 to 6 years, and are all reported to be satisfactory, as far as water-tightness and the general condition of the water-proofing are concerned.

With reference to the use of burlap in the water-proofing membrane, the speaker thoroughly agrees with Mr. Wagner's statement in Appendix B, that it should never be used in the untreated or raw state, and he has doubts as to the possibility of overcoming its bad properties by treatment. Treated cotton fabric seems to be preferable.

Referring to Mr. Wagner's specifications for water-proofing, the speaker offers the following comments:

Paragraph 1.—"Depth", in this paragraph, refers to the depth of steel or concrete construction. It would seem rather more logical, in a specification for water-proofing, to refer to the depth required for the water-proofing itself. In the speaker's practice it has been found that one can work with a minimum of 2 in. for the water-proofing itself plus the protective covering, using 1½ in. of reinforced cement mortar for the protective coating; and with a steel-plate floor,

4 to 4½ in. from the top of the plate to the top of the protection, 4 in. with cement mortar protection, and 4½ in. with brick. Mr. Carpenter.

Paragraph 2.—For providing a drainage slope, the speaker has successfully used reinforced cement mortar with a minimum thickness of 1½ in.

Paragraph 3.—The speaker has thought the closed form of drains specified (and illustrated) by Mr. Wagner rather objectionable for bridges in cold climates, and for that reason he has used inlets leading to open gutters.

One point that Mr. Wagner illustrates but does not mention in his specification, is that it is not a good thing to put the drains or inlets directly under the tracks. A number of the earlier bridges had that construction, and it was difficult to get at the drains to clean them.

Paragraph 4.—This paragraph provides, in effect, for continuous girder construction, which in some cases would be unsatisfactory.

Paragraph 5.—The necessity for this paragraph is not apparent, and in some cases the speaker would not want to use gusset-plates without angle reinforcement on the outer edges.

Paragraph 9.—The speaker would prefer to specify reinforced felt or treated cotton fabric in place of treated burlap.

Paragraph 10.—The thickness of reinforced cement protection may be reduced to 1½ in. as a minimum.

Paragraph 12.—For insurance of absolute water-tightness, caulked edges of the steel at joints in the steelwork are to be recommended, although expensive. In addition to such caulking, certain openings would generally require caulking with lead, oakum, or other suitable material.

Paragraph 13.—Reinforced cement mortar, composed of 1 part cement and 3 parts sand, minimum thickness 1½ in., has been the speaker's standard for steel-plate decks.

Paragraph 15.—Coal-tar paints would be preferable with coal-tar pitch water-proofing. The necessity for painting mortar surfaces, especially where coal-tar pitch is to be used, is not apparent.

Paragraph 18.—Concrete or mortar surfaces should be dry when paint or water-proofing is applied. Presumably, the treatment recommended is intended to insure dryness, but might not.

Paragraph 39.—The criss-cross form of laying water-proofing is novel to the speaker. The requirement that the first layer of waterproofing felt be not cemented to the bridge floor seems to be of doubtful utility; the opposite practice has obtained in all the speaker's bridges.

Some other points of criticism of the specifications are covered by the general discussion and in the specifications used in the speaker's work, herewith submitted.

Mr. Carpenter. Figs. 15 to 19 are reproductions of drawings for different types of water-proofing applied to bridge structures. Commenting on the latter:

Fig. 15 shows a floor with track ties set in concrete, and no other water-proofing. The openings along the girders leave the gutters accessible from above, and the floor connections open for inspection. They also cause some slight saving in load on the girders. The ¼-in. plate-covers prevent rubbish from falling into the openings and clogging the gutters.

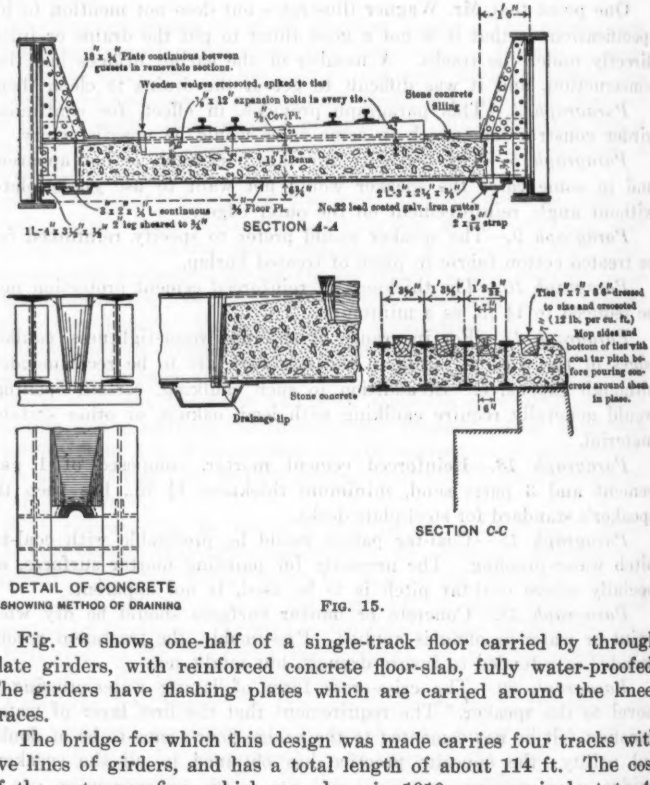


FIG. 15.

Fig. 16 shows one-half of a single-track floor carried by through plate girders, with reinforced concrete floor-slab, fully water-proofed. The girders have flashing plates which are carried around the knee-braces.

The bridge for which this design was made carries four tracks with five lines of girders, and has a total length of about 114 ft. The cost of the water-proofing, which was done in 1910, was equivalent to 46 cents per sq. ft. of water-proofed area, this cost including the reinforced concrete floor-slab, the water-proofing proper, and the brick and concrete protection, but not including the girder flashing and the

Mr. Carpenter.

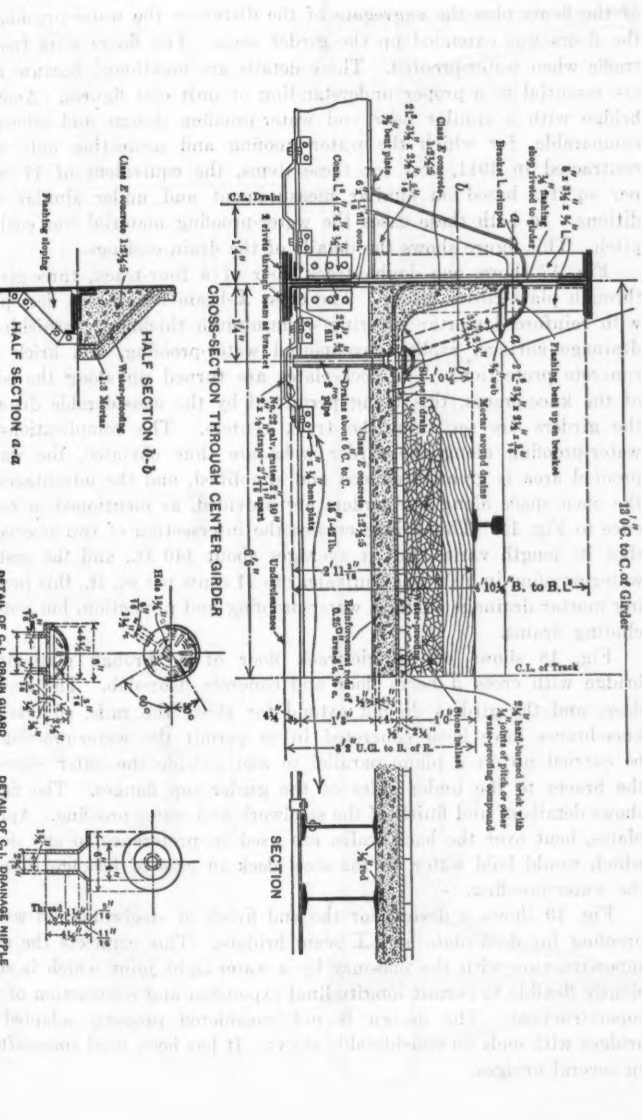


FIG. 16.

Mr. Carpenter.

drains. The width of water-proofed area was taken as the full width of the floors plus the aggregate of the distances the water-proofing of the floors was extended up the girder webs. The floors were free of traffic when water-proofed. These details are mentioned because they are essential to a proper understanding of unit cost figures. Another bridge with a similar floor and water-proofing design and otherwise comparable, for which the water-proofing and protection only were contracted in 1911, cost for these items, the equivalent of 17 cents per sq. ft., based on similar measurement and under similar conditions. In both these cases the water-proofing material was coal-tar pitch. This figure shows the details of the drain castings.

Fig. 17 shows one double-track floor of a four-track, three-girder, through plate-girder bridge. The cross I-beam floor has a deck-plate with reinforced mortar covering of minimum thickness, furnishing a drainage surface, asphaltic-compound water-proofing, and brick and concrete protection. The deck-plates are turned up along the edges of the knee-braces, this being permitted by the considerable distance the girders are set from the track centers. The complications of water-proofing along the girder webs are thus obviated, the water-proofed area is greatly reduced and simplified, and the advantages of the open space along the girders are provided, as mentioned in reference to Fig. 15. This bridge crosses the intersection of two streets, so that its length varies, but it averages about 140 ft., and the cost of water-proofing in 1909 was equivalent to 21 cents per sq. ft., this including mortar drainage surface, water-proofing and protection, but not including drains.

Fig. 18 shows one single-track floor of a through plate-girder bridge with cross I-beam floor and concrete floor-slab. The floor is deep, and the girders do not extend far above the rails, so that the knee-braces have been concreted in to permit the water-proofing to be carried up on a plane parallel to and outside the outer edges of the braces to the under sides of the girder top flanges. The figure shows details of and finish of the steelwork and water-proofing. Apron-plates, bent over the back walls, are used in preference to any detail which would hold water on the steel deck in case of leakage through the water-proofing.

Fig. 19 shows a design for the end finish of steelwork and water-proofing for deck-plate and I-beam bridges. This connects the steel superstructure with the masonry by a water-tight joint which is sufficiently flexible to permit longitudinal expansion and contraction of the superstructure. The design is not considered properly adapted to bridges with ends on considerable skews. It has been used successfully on several bridges.

Mr. Carpenter.

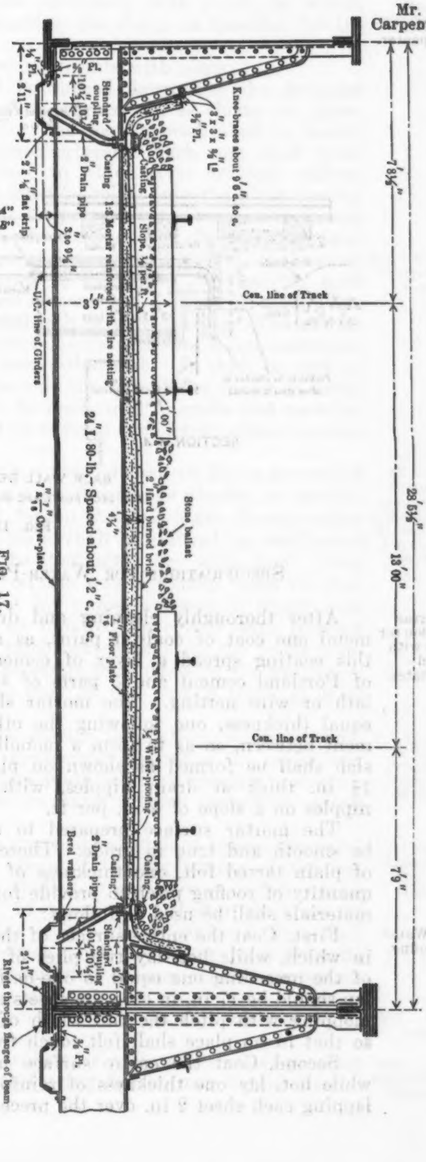


FIG. 17.

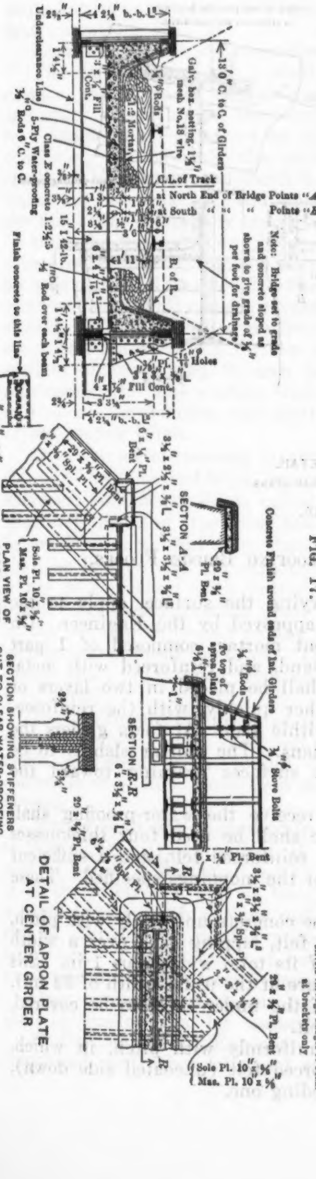


FIG. 18.

Mr. Carpenter.

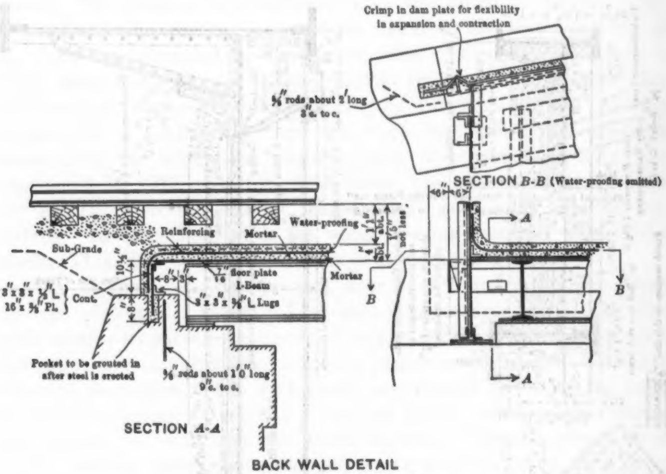


FIG. 19.

SPECIFICATIONS FOR WATER-PROOFING BRIDGE FLOORS.

I.—Mortar Foundation for Bridges with Steel Deck Plates.

After thoroughly cleaning and drying the surface, apply to the metal one coat of coal-tar paint, as approved by the Engineer. On this coating spread a layer of cement mortar, composed of 1 part of Portland cement and 3 parts of sand, and reinforced with metal lath or wire netting. The mortar shall be placed in two layers of equal thickness, one following the other quickly, with the reinforcement between, so as to form a monolithic slab. At facia girders the slab shall be formed as shown on plans. The mortar slab is to be 1½ in. thick at drain nipples, with surfaces draining toward the nipples on a slope of ⅛ in. per ft.

The mortar surface prepared to receive the water-proofing shall be smooth and true to grade. There shall be used four thicknesses of plain tarred felt, one thickness of reinforced felt, and a sufficient quantity of roofing pitch to provide for the moppings specified. These materials shall be used as follows:

II.—Water-Proofing.

First. Coat the entire surface of the concrete uniformly with pitch, in which, while hot, lay two plies of felt, lapping each over a width of the preceding one equal to one-half its total width plus 1 in. (this lap should be 17 in. in case the sheets are of the usual width of 32 in.), mopping with pitch the full width of the under sheet to be covered, so that in no place shall felt touch felt.

Second. Coat the entire surface uniformly with pitch, in which, while hot, lay one thickness of reinforced felt (uncoated side down), lapping each sheet 2 in. over the preceding one.

Mr. Carpenter.

Third. Coat the entire surface uniformly with pitch, in which, while hot, lay two plies of felt, lapping the sheets as specified for the first operation.

Fourth. Coat the entire surface uniformly with pitch.

The felt water-proofing shall be turned down into the drainage nipples, and at facia girders shall be turned up, as shown on plans, and stuck closely to the steelwork. The roofing pitch shall be heated to a temperature approved by the Engineer. Each coat shall cover completely and entirely the surface to which it is applied, without cracks or blow-holes, and the felt must be rolled into the pitch coating while the latter is hot, and must be pressed into it so as to insure the felt being completely stuck to the coating over the entire surface.

III.—Mortar Protection Course.

Over the water-proofing layer thus formed shall be laid a protection course of reinforced cement mortar, 1½ in. thick. The mortar shall be composed of 1 part of Portland cement and 3 parts of sand. The reinforcement shall consist of metal lath or wire netting. The mortar shall be placed in two simultaneous layers with the metal reinforcement between so as to form a monolithic slab. It shall be laid in sections of about 9 sq. ft. area, to suit the size of the sheets of metal reinforcement. The surface shall be made true to grade and troweled to a smooth hard finish, and shall be covered completely with a coating of hot roofing pitch.

IV.—Brick Protection Course (to be Substituted for Mortar Protection Course if Specified).

Over the water-proofing layer thus formed shall be laid a protection course of hard-burned brick, except around drainage nipples, on vertical surfaces, and otherwise as called for on the drawings. Brick, unless absolutely dry and free from moisture, shall be stacked up and heated before laying, so that they shall be thoroughly dry when laid. They shall be laid on the flat directly on the water-proofing, with as close joints as consistent with provision for proper subsequent filling with pitch. The water-proofing shall be followed up closely with pouring of the joints with heated roofing pitch, which shall fill the joints completely to the top of the brick. The entire top surface of the protection course shall be mopped with one coat of hot roofing pitch at least ⅜ in. thick.

V.—Thickness of Water-Proofing on Deck-Plate Bridges.

The maximum thickness of the mortar foundation, water-proofing, and protection course shall not appreciably exceed 4 in. under any track, measuring from top of deck-plate when a mortar protection course is used, nor more than 4½ in. when a brick protection course is used.

VI.—Time for Materials to Dry and Harden.

Mortar shall not be applied to any painted surface before the paint is thoroughly dry, or within 24 hours of the application of the last coat of paint. Pitch shall not be applied to any mortar surface unless it is absolutely dry and at least 24 hours old. Mortar shall not be applied to any pitch surface unless it is perfectly cool. Ballast shall not be laid on any mortar protection until the latter is at least 24 hours old. Tracks shall not be supported on any mortar protection until the latter is at least 7 days old.

VII.—Workmen.

Only competent workmen, especially skilled in this kind of work, shall be employed to lay water-proofing.

VIII.—Other Water-Proofing Materials.

If directed by the Engineer, asphalt or some other water-proofing compound, and other felts and reinforcing fabric, may be substituted

Mr. Carpenter. for the roofing pitch and felt hereinabove mentioned. In such case the Company shall pay to the Contractor the excess cost, if any, of such materials, over the cost of the materials specified.

IX.—Roofing Pitch. Roofing pitch shall be the best American, straight-run, coal-tar pitch, with a melting point of not less than 130 nor more than 140° Fahr., and shall have a specific gravity of not less than 1.23 at 60° Fahr., and an evaporation not greater than 8½% after 7 hours heating at 275° Fahr., and shall be otherwise equal to that furnished by ——.

X.—Asphalt. Asphalt shall be pure, refined, high-grade asphalt, free from coal-tar, turpentine, or any of their products, and shall contain not less than 97% of bitumen, soluble in carbon-bisulphide; and, of the bitumen thus soluble in carbon-bisulphide, not less than 60% shall be soluble in 88° naphtha, air temperature. It shall not volatilize more than 5% under a temperature of 350° Fahr., in 7 hours. It must not be affected by a 25% solution of sulphuric acid acting upon it at ordinary room temperature for 7 hours. It shall not flow under a temperature of 212° Fahr. and shall not become brittle at 15° below zero Fahr., when spread in a layer ⅛ in. thick, on thin glass.

XI.—Other Water-Proofing Compound. Water-proofing compounds, felts, and fabrics, other than those specified herein, shall be of quality approved by the Engineer.

XII.—Cement, Sand, and Mortar. Cement, sand, and mortar, shall be as required by the Standard Specifications for Concrete Masonry.

XIII.—Brick. Brick shall be dense, hard-burned, of uniform size and quality, with square corners and plane sides, and shall not increase in weight more than 10% when immersed in water for 7 hours. The brick shall not be more than 2 in. thick. The lateral dimensions shall be satisfactory to the Engineer.

XIV.—Tarred Felt. Tarred felt shall be saturated felt weighing not less than 14 lb. per 100 sq. ft., and shall be equal to that known as ——, manufactured by ——.

XV.—Reinforced Felt. Reinforced felt shall be equal to that known as —— and manufactured by ——.

XVI.—Metal Reinforcement. Metal reinforcement for the protection course shall be No. 27 gauge expanded-metal lath, No. 18 gauge 1½-in. mesh wire cloth, or other equivalent reinforcement.

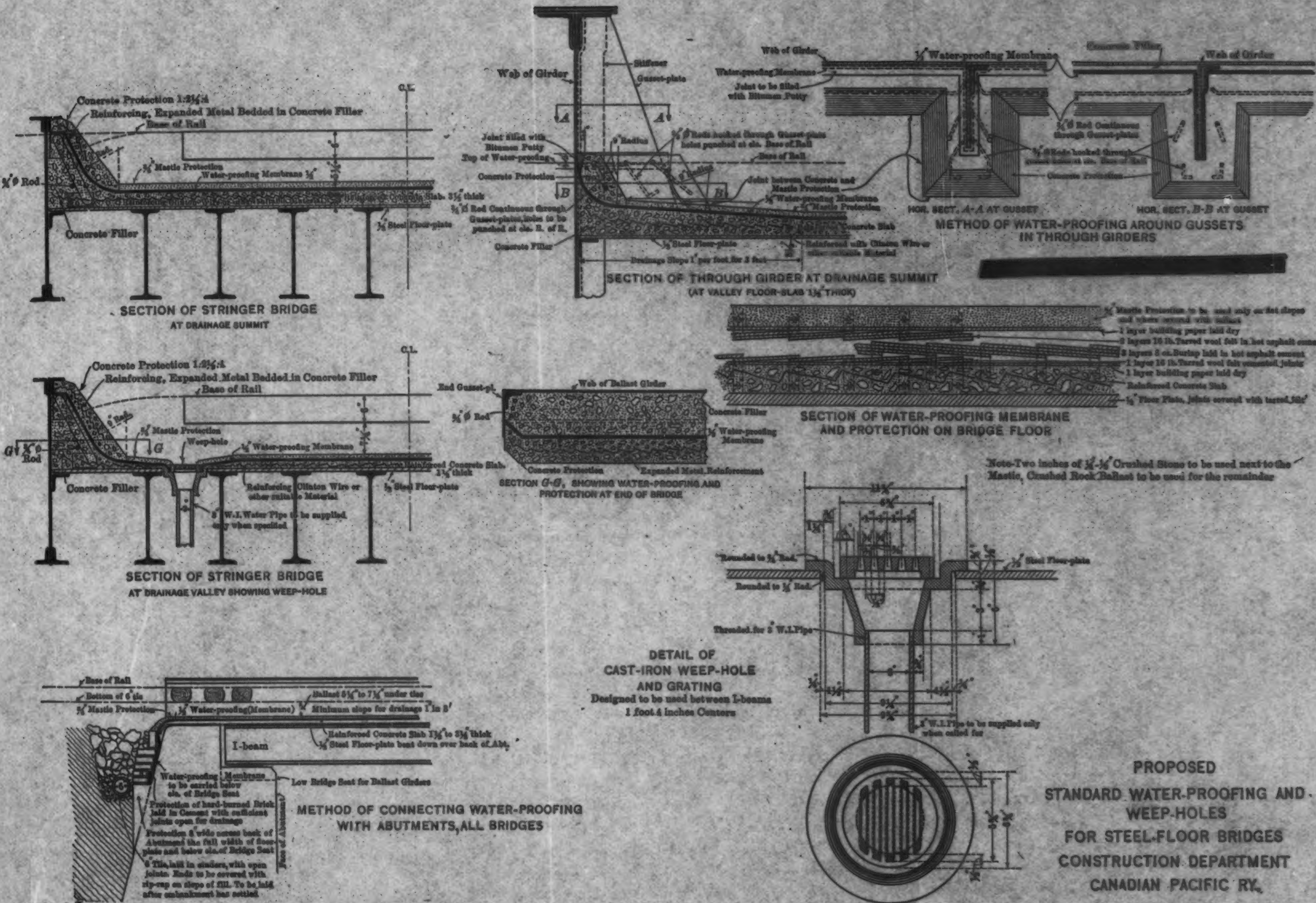
Mr. DeLamere. C. T. DeLamere,* Assoc. M. Am. Soc. C. E. (by letter).—The Canadian Pacific Railway Company is building a branch through the Cities of Montreal and Maisonneuve, in the Province of Quebec. On this branch thirteen solid-floor steel bridges over streets have been completed with water-proofing, and eight are still under construction.

These thirteen structures were water-proofed under varying weather conditions, and were exposed to severe cold (28° below zero) and extreme heat (125° air above the mastic) before the ballast was placed on the deck; thus affording an excellent opportunity to observe the water-proofing under different conditions.

Certain changes were suggested as a result of this work, and a proposed standard and drainage diagram for the unfinished work was

* Montreal, Que., Canada.

PLATE VIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXIX, No. 1838.
DELAMERE ON
WATER-PROOFING RAILROAD BRIDGE FLOORS.



PROPOSED
STANDARD WATER-PROOFING AND
WEEP-HOLES
FOR STEEL-FLOOR BRIDGES
CONSTRUCTION DEPARTMENT
CANADIAN PACIFIC RY.

made up, Plate VIII and Fig. 20, although not yet approved. Attention is called to the following features. **Mr. DeLamere.**

1.—The introduction of a thin slab of reinforced concrete between the steel floor-plate and the membrane will act as an insulator, protecting the membrane from the violent temperature changes to which it would be subjected if in direct contact with the steel floor-plate.

2.—The membrane is laid with a steep pitch, to shed the water as quickly as possible; 1 in. in 8 ft. is recommended.

3.—The membrane should be pitched, so as to throw the water away from the angle between the floor and the side. Particular attention is paid to this feature on half-through girders with gusset-plates.

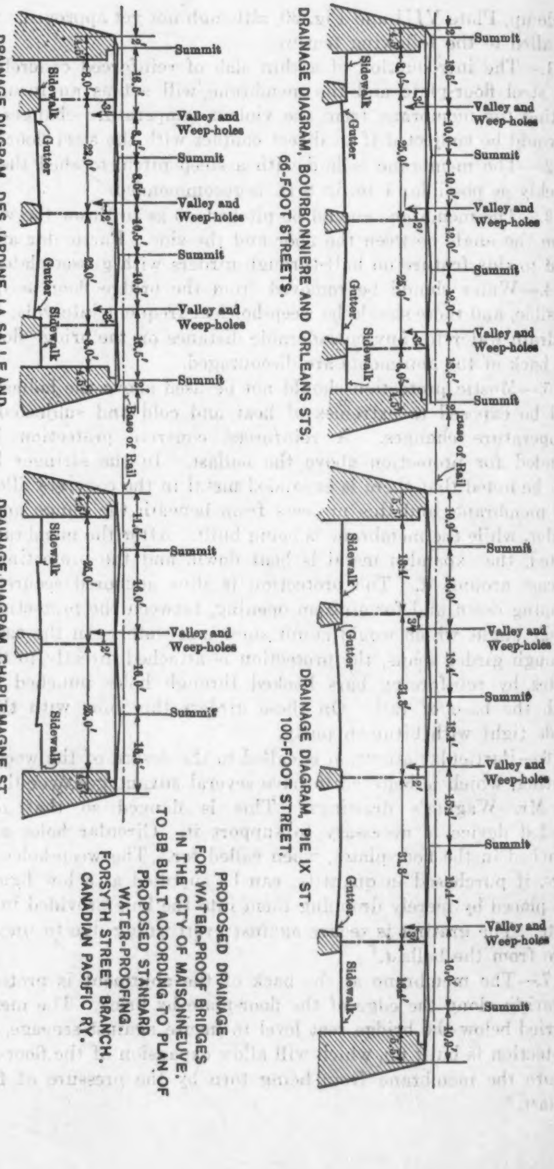
4.—Water should be removed from the bridge floor as quickly as possible, and there should be weep-holes at frequent intervals. Attempts to drain water for any considerable distance on the bridge floor or over the back of the abutments are discouraged.

5.—Mastic protection should not be used above the ballast where it will be exposed to extremes of heat and cold and subjected to rapid temperature changes. A reinforced concrete protection is recommended for protection above the ballast. In the stringer bridges it will be noted that there is expanded metal in the concrete filler beneath the membrane, and this projects from beneath the flange angle of the girder, while the membrane is being built. After the membrane is completed, the expanded metal is bent down, and the protecting concrete is cast around it. The protection is thus anchored securely against slipping down and forming an opening, between the protection and the flange angle, which would admit snow and water. In the case of half-through girder spans, the protection is attached directly to the gusset-plates by reinforcing bars hooked through holes punched on a line with the base of rail. On these girders the joint with the steel is made tight with bitumen putty.

6.—Particular attention is called to the design of the weep-hole and grating, which is believed to have several advantages over those shown in Mr. Wagner's drawings. This is flanged so that no special riveted device is necessary to support it. Circular holes are merely punched in the floor-plates, when called for. The weep-holes and gratings, if purchased in quantity, can be supplied at a low figure. They are placed by merely dropping them into the hole provided in the floor-plate. The grating is secure against overturning due to unequal pressure from the ballast.

7.—The membrane at the back of the abutment is protected from shearing along the edge of the floor-plate by brick. The membrane is carried below the bridge seat level to insure against seepage, and brick protection is built up, which will allow expansion of the floor-plate, and secure the membrane from being torn by the pressure of filling and ballast.

Mr. DeLamere.



PROPOSED DRAINAGE FOR WATER-PROOF BRIDGES IN THE CITY OF MAISONNEUVE TO BE BUILT ACCORDING TO PLAN OF PROPOSED STANDARD WATER-PROOFING FORSYTH STREET BRANCH. CANADIAN PACIFIC RY.

FIG. 20.

Fig. 21 shows the method (used successfully) of securing the mastic protection to the concrete, expanded metal reinforcement being embedded in the concrete filler. Difficulty had been caused by the mastic slipping on the membrane during hot weather. In moulding this concrete filler, a slot was cast along the upper edge of the surface to be water-proofed. Expanded metal was embedded in the concrete, and projected about 15 in. from this slot. The metal was bent up out of the way, and the membrane was laid running into the slot. After the first coating of mastic protection, the expanded metal was bent down and **Mr. DeLamere.**

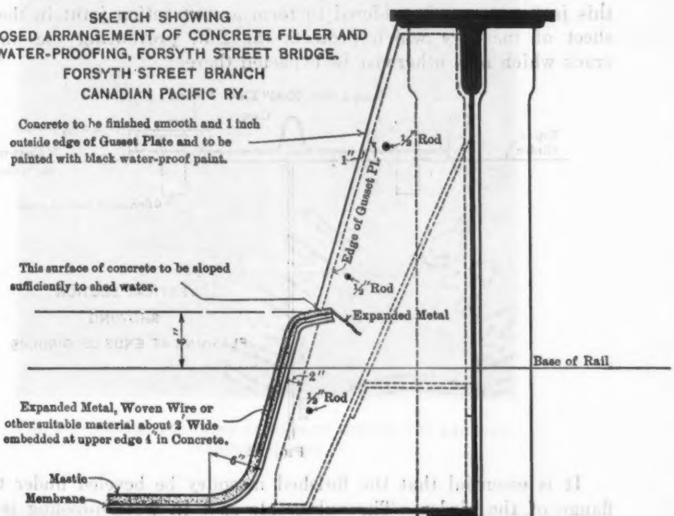


FIG. 21.

secured with a few roofing nails. Mastic was plastered over, and the expanded metal was entirely embedded in it. The mastic is thus secured against slipping down in hot weather. However, the use of mastic is not recommended for exposed work in the climate of Montreal.

JOHN JERVIS VAIL,* ASSOC. M. AM. SOC. C. E. (by letter).—In waterproofing half-through bridges in elevating the Pennsylvania Railroad at Rahway, N. J., especial pains have been taken to exclude water where ends of girders meet. At such places, where girders come **Mr. Vail.**

* Rahway, N. J.

Mr. Vail. together over columns, rain may enter and corrode the steel, penetrating under water-proofing, perhaps, and doing damage elsewhere, also. Fig. 22 shows the protective copper flashing devised by H. R. Leonard, M. Am. Soc. C. E., Engineer of Bridges, and R. Farnham, Jr., M. Am. Soc. C. E., Assistant Engineer of Bridges.

The sheet-metal, crimped over the joint at the top of the girders, is carried down the whole height of the girders, inside and outside. On the inside another jacket is placed, after the water-proofing membrane is complete, to shed water away from any break in the felt caused by contraction cracks in the masonry over the columns. To this jacket copper is soldered to form a contraction joint in the 2½-in. sheet of masonry which protects the felt, preventing the unsightly crack which may otherwise be expected there.

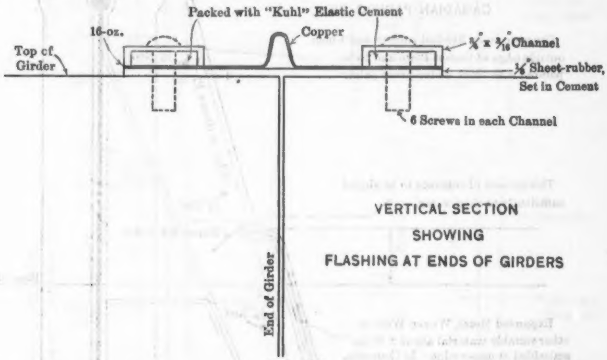


FIG. 22.

It is essential that the finished masonry be beveled under the top flange of the girder. The vulnerable spot in water-proofing is where the membrane is finished against the web of the girder. This attachment cannot be relied on to be water-tight, especially around angles and rivets. A bridge finished with masonry flush with the edge of the girder admitted rain water, driven by storm, into the minute space between the concrete and the under side of the top of the girder. It found its way behind the water-proofing, dripping out along the girder, under the bridge floor. A chamfer, cut as shown in the drawing, effectually shed the water, and made the bridge tight. To place the concrete as close as possible against the under side of the top of the girder, the forms are filled an inch or two above the girder. When the masonry has set, the surplus is cut away with a chisel and bush-hammer.

It would seem that if any water at all passes the water-proofing devices, the design is not a success. It cannot be known whether the

FIG. 23.—COPPER FLASHING AT ENDS OF GIRDER, TOP AND SIDE. INSIDE OF GIRDER.

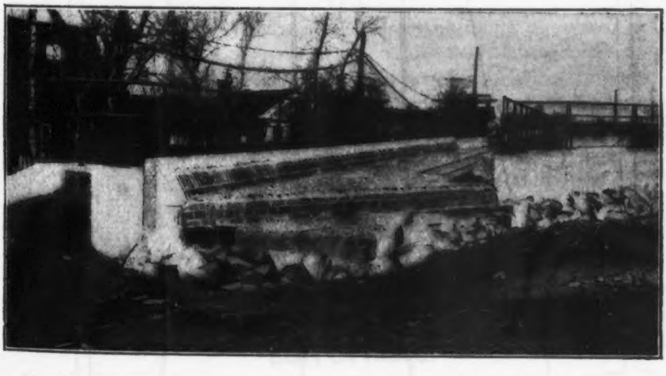
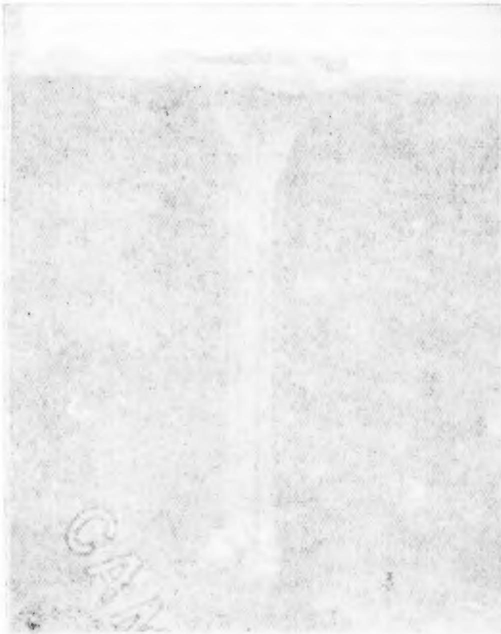
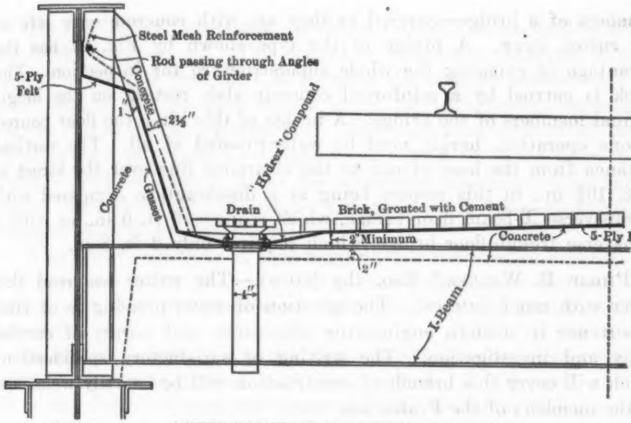


FIG. 24.—REINFORCED CONCRETE SLAB FLOOR, NOT IN CONTACT WITH GIRDERS.

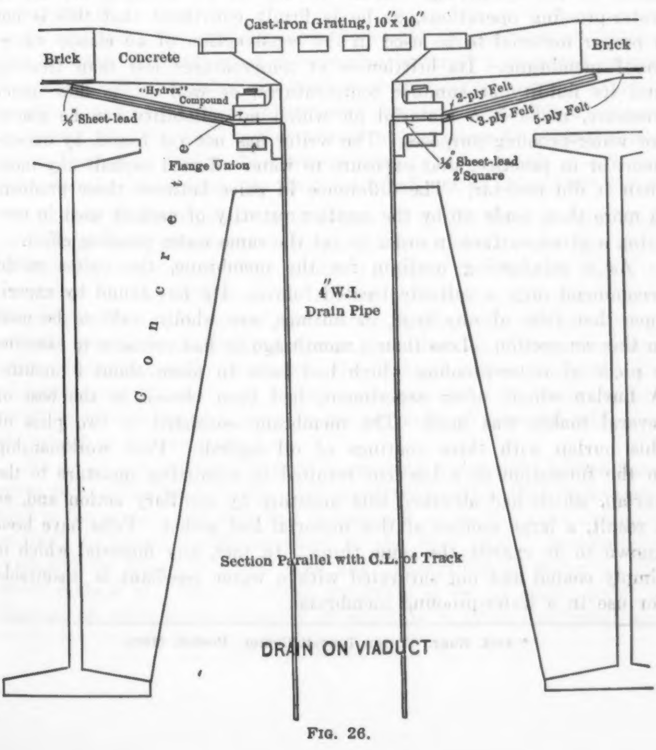


Mr. Vail.



WATER-PROOFING HALF-THROUGH VIADUCT

FIG. 25.



DRAIN ON VIADUCT

FIG. 26.

Mr. Vail. members of a bridge—covered as they are with concrete—are safe or are rusted away. A bridge of the type shown by Fig. 24 has the advantage of exposing the whole superstructure for inspection. The track is carried by a reinforced concrete slab, resting on the longitudinal members of the bridge. A bridge of this kind, the floor poured in one operation, hardly need be water-proofed at all. The vertical distance from the base of rail to the clearance line over the street is 3 ft. 10¾ in., in this respect being at a disadvantage compared with a transverse I-beam floor bridge, which requires 3 ft. 0 in., or with a transverse trough-floor bridge, which requires only 2 ft. 8 in.

Mr. Walker. PHILIP B. WALKER,* ESQ. (by letter).—The writer has read this paper with much interest. The question of water-proofing is of vital importance in modern engineering structures, and worthy of careful study and investigation. The writing of satisfactory specifications which will cover this branch of construction will be heartily welcomed by the members of the Profession.

The writer wishes to commend the author for the stand he has taken in not specifying coal-tar or any of its products for use in water-proofing operations, as he is firmly convinced that this is not a proper material to be used in the construction of an elastic water-proof membrane. Its brittleness at temperatures less than freezing and its fluidity at summer temperatures, as well as its flow under pressure, make it a material on which no reliability can be placed for water-proofing purposes. The writer has not yet found, by experiment or in practice, that exposure to water affected asphalt any more than it did coal-tar. The difference in price between these products is more than made up by the smaller quantity of asphalt used in covering a given surface in order to get the same water-proofing effect.

As a reinforcing medium for the membrane, the writer would recommend only a suitably treated fabric. He has found by experience that felts of any type, or burlaps, are wholly unfit to be used in this connection. Less than a month ago he had occasion to examine a piece of water-proofing which had been in place about 9 months. A burlap which, after experiment, had been chosen as the best of several makes was used. The membrane consisted of two plies of this burlap with three coatings of oil asphalt. Poor workmanship in the formation of a lap had resulted in admitting moisture to the burlap, which had absorbed this moisture by capillary action and, as a result, a large section of this material had rotted. Felts have been known to do exactly the same thing. In fact, any material which is simply coated and not saturated with a water repellant is unsuitable for use in a water-proofing membrane.

* Asst. Engr., Boston Transit Comm., Boston, Mass.

The base of the fabric used should be a closely woven drill weighing about 12 oz. per sq. yd. This should be completely saturated with an asphalt similar to that used in forming the finished membrane. Not every asphalt can be used with a given fabric, therefore experiment should determine the proper one. Although each individual thread forming the fabric must be completely saturated, the space between the threads must be open to allow the successive moppings of asphalt to unite. The drill used should be woven so as to have equal tensile strength lengthwise and crosswise. Mr. Walker.

A membrane made up of such a fabric will have considerable stretch and will not crack or tear, even under severe temperature changes. The use of such a fabric permits of tight joints around gussets, stiffeners, etc.

In regard to the asphalt, the writer would consider that a smaller range in the melting-point might be specified. A difference of from 20 to 25° Fahr. between the maximum and minimum melting-points ought to cover the various conditions as found on the work and yet give an asphalt less variable in those qualities which the melting-point controls.

In specifying the melting-point, he would suggest that the author give in detail the method of determining it, until a standard method is accepted. When stating the method of application and the quantity of asphalt to be used, it would be more complete if the following were included: "Asphalt shall not be heated at any time to a temperature higher than 450° Fahr. The temperature of the asphalt in the pail just before being mopped into the fabric shall not be below 380° Fahr."

CHARLES RUFUS HARTE,* M. AM. SOC. C. E. (by letter).—Mr. Wagner's paper deals admirably with a subject which to some may seem of little real importance, but which, as many an engineer knows to his sorrow, is anything but the simple proposition it appears to be at first glance. As a general proposition, little really serious damage actually results from faulty drainage, but there is always the possibility of dangerous developments, the leakage causes unsightly discoloration, and, if over busy passageways, may drop on passers-by and result, in the course of a year, in the payment of a considerable sum for ruined or injured garments. Mr. Harte.

No small part of the difficulty in securing water-tight bridge floors arises from a failure to realize that capillarity rather than hydrostatic head is the enemy, and that fine cracks well up on the sides are quite as likely to cause trouble as those in the floor itself. Incidentally, it should be borne in mind that the appearance of water on the under surface of a steel or stone structure is by no means proof of leakage; an astonishing quantity of moisture is often condensed and,

* New Haven, Conn.

Mr. Harte.

creeping to low points, gives there the impression of a serious breakdown of the water-proofing.

This creeping of water, whether from leaks or condensation, may be most annoying. In discussing a previous paper* by Mr. Wagner, the writer described a case on the New York, New Haven and Hartford Railroad, at Boston, where water followed the wire supports and slightly convex spanners over the edge of the collecting trough which they supported, and then dropped on the unfortunate passer-by. The spanners were made concave and then the drip went into the trough. This experience indicates the possibility of leading stray water to the main conductors by very small troughs, but the writer is not aware of any actual construction following this line.

Conditions, under almost any circumstances, are exceedingly favorable to obstruction, making it very important to give the closed pipes generous sizes and to provide catch-basins to trap the more than "fifty-seven varieties" of rubbish that tend to cause stoppage. A stone viaduct of several arches, water-proofed with a five-ply felt and tar membrane carried well up on the spandrel walls, was drained by 4-in. iron pipes. These had several light bends, and were led from the dry, coarse sand of the foundation bed through the piers, and continued about 2 ft. above the valleys, the upper portion being perforated with an extensive series of ½-in. holes. In less than 6 months, the arches began to weep heavily. The perforated pipes had been placed between the tracks in order to meet just this contingency, but their distance below grade and the heavy traffic combined to make the work difficult and expensive. The pipes were rodded out, and perforated manholes were built over them. This made it much easier to rod the passages, but, after a number of exasperating stoppages and consequent weeping on the inside of the arches, a well-driller was set up and an 8-in. hole was drilled straight through to the gravel, thus ending the trouble, but not before the weepage had unpleasantly marked the intrados with lime deposits. Conditions behind an abutment will rarely be as favorable as in this case, and, unless the main piping is nearly vertical, cinders and the like, which wash in, soon choke it. Mr. Wagner's statement that it is "inadvisable" to throw all the water behind the back walls may be criticized only on the ground that it is not strong enough.

The drainage should be by the straightest and shortest path. The viaduct described was over a parkway; a similar structure designed by the writer, which spanned a stream, has 10-in. discharge passages from the bottoms of the catch-basins through the ring near the springing line. Perforated manholes over the basins admit the water, and would give access if necessary, but thus far there has been no

* "The Elevation of the Tracks of the Philadelphia, Germantown and Norristown Railroad, Philadelphia, Pa.," *Transactions*, Am. Soc. C. E., Vol. LXXVI, p. 1900.

trouble—nor, in view of the size of the passage, is this particularly surprising. Although neither of these structures is of the "solid steel-floor" class discussed by the author, there are many elements of the drainage problem in common. Mr. Harte.

Bridges over streets obviously cannot obtain such direct discharge as the river bridge described. Where there is little frost, spans with curb columns can often be drained satisfactorily by leaders, in the columns, to the gutter; if there is likelihood of much weather at temperatures below freezing, such leaders should be run as directly as possible to the sewer. In any case, sizes less than 6 in. in diameter are hardly large enough. Runs should be straight, and connections should be made by Y or T, the blank leg having a readily removable cover, through which, if necessary, a stout clearing rod can be run. Stiff wire will take easy bends, but a "big stick" will do the work in a fraction of the time. It is essential, of course, that the use of a clearing rod be borne in mind in placing the piping in the first instance, and where a bend at an inaccessible point is unavoidable, a bedding of concrete will permit of reasonable rodding without danger of punching through.

With wide track spacing, manholes and the like at grade will naturally be placed between tracks. With close track spacing and frequent trains, it is usually better to place them in the center of a track the use of which may be discontinued when necessary, as this will cause less actual interference and danger than if they are placed between the tracks.

These points are perhaps a little beside the main subject of the paper, but they are so closely related, and, although perfectly obvious after attention has been called to them, are so often overlooked in design that it has seemed desirable to the writer to include them here.

Samuel Tobias Wagner,* M. Am. Soc. C. E. (by letter).—In closing the discussion on this paper, the writer first desires to express his appreciation of the valuable ideas brought out by those interested in the subject. He regrets, however, that there has not been a freer expression of opinion on the subject of materials by those who have had experience with asphalt and coal-tar pitch. Mr. Wagner.

Before commenting on the opinions expressed in the discussion, he desires again to emphasize the fact that the scope of the paper is rather narrow, as it covers only that type of water-proofing applicable to railroad bridges with solid floors. The general trend of the discussion, fortunately, has been confined to these lines, or where it has digressed it has been within safe limits.

*Philadelphia, Pa.

Mr. Wagner.

The writer has rewritten the specifications presented in the paper, in the light of the discussion, making such changes as in his opinion are warranted by the additional information contributed.

The closing discussion has been arranged under the following headings, as it is believed that they will be better than any attempt to confine the remarks to specific clauses in the specifications:

1.—General Clauses.
2.—Details of Construction.
3.—Materials.
(*a*) Felts and Fabrics.
(*b*) Asphalts.
(*c*) Coal-Tar Pitch.
(*d*) Asphalt Mastic.

1.—General Clauses.

There can be no question that really satisfactory water-proofing cannot be done in cold or bad weather, and every effort should be made to avoid the selection of such times for doing it. Mr. Himes' suggestions are along the right lines. If the time for doing the work can be controlled, it should be done by all means, but the writer's experience has been different, and Paragraph 39 has been written to control the time as far as possible.

It has been found to be especially difficult to lay any bitumen in cold weather or on cold materials and secure good adhesion where such is necessary; this is most markedly the case with coal-tar pitch, which hardens so quickly and becomes so brittle below the freezing point as to make adhesion almost impossible. The asphalts with low melting points and high ductilities appear to be the best for use under such conditions.

Another reason for water-proofing solid-floor railroad bridges, in addition to those given in the paper, has been brought out by Mr. Auryansen when he calls attention to the question of electrolysis. Such bridges are likely to be used on steam roads in urban territory, on which it is almost certain that electric power is likely to follow, and engineers should do all in their power to keep stray currents from acting on the metal in the floors.

The writer thoroughly agrees with Mr. Vail that if any water at all passes the water-proofing devices, the design is not a success. Although, of course, it is very unfortunate to see water dripping from the floor of a bridge, it is worse to know that it is leaking at the top and causing unknown trouble by corrosion of the steelwork.

Mr. Carpenter agrees with the writer that the ballast is one of the main reasons for requiring water-proofing, but does not agree with him that it is one of the things which can be dispensed with. If there is any reason, from a railroad standpoint, for the existence of

a solid-floor bridge, it is believed that it is because it carries a standard track like the rest of the permanent roadbed. **Mr. Wagner.**

Mr. Ray realizes that there is no especial difficulty in water-proofing a deck structure, and the writer is glad to note that he also believes it is wise to water-proof any structure where water can readily get into the concrete. For many years the writer has felt that, in reinforced concrete structures subject to the action of water, water-proofing should be used. It is believed that the protection given is well worth the additional cost.

The writer wishes to thank Mr. Ray for again calling attention to the fact that, when water-proofing is to be applied to a solid-floor bridge of any type, the kind of materials to be used should be determined before the details of the steel construction are fixed.

The best method of contracting for water-proofing is a debatable question. In the writer's experience, the aim in recent years has been to arrange so that the contract is made directly with a water-proofing contractor who is directly interested in the quality of the work, and allow him to sub-let the concrete, drainage, etc. From the standpoint of a railroad, this method is believed to secure the best results by having the contractor directly interested in the most important part of the work. Any arrangement which makes a sub-contractor of the man who supplies and places the water-proofing material has been found to remove correspondingly his interest in the best materials and workmanship. Originally, the company with which the writer is connected made a contract with a steel manufacturer for the whole bridge superstructure, including the water-proofing. This frequently resulted in the sub-contractor for the water-proofing doing as little work as he could and showing no interest in its quality.

2.—Details of Construction.

The writer agrees with Mr. Himes as to the disadvantageous qualities of the shallow floor without ballast. A solid floor carrying the standard ballasted roadbed is an ideal construction, from the standpoints of both maintenance and public comfort, and these are the real reasons for its existence, because, otherwise, the additional cost would not be warranted. The use of ballast, particularly when dirty, has one drawback: it holds the water, and, on long and level structures, makes the question of the disposition of the water a more complicated problem than if there was no ballast. It would be interesting to know what was in Mr. Himes' mind when he expressed a preference for sand in place of stone for ballast.

The writer agrees with Mr. Carpenter that the carrying of the water-proofing over the top flanges of low half-through girders is not advisable if the distances from center to center of girders must be increased.

Mr. Wagner.

The details given in Mr. Auryansen's discussion are very valuable, and show how such details could be studied for each and every case. The writer can understand why coal-tar pitch will give good results if the flashings shown have been used. No dependence whatever is placed on the adhesive qualities or ductility of the material. The detail shown in Fig. 12 is a good one, and has been used by the writer on a number of bridges with good results; but it cannot be applied to girders where the distance from top of rail to top flange is great.

Any "sloping hood" or "flashing-angle", such as referred to by Mr. Quimby, over the joint along a through girder, will interfere seriously with the proper preparation of the joint, and, unless it is fitted up tightly around the gussets and stiffeners, which operation is both very difficult and expensive, it will add nothing to the efficiency of the joint and cause much difficulty in making repairs, if the life of the asphalt proves to be shorter than that of the other materials.

The use of flashing-angles on the main girder webs may be justified if they are efficient, and also if the cost per pound of structural steel is not too high, but the detail is certainly a very expensive one, and its use should be carefully considered.

Mr. Quimby asks why the drainage of the water on to the bridge seat is a bad detail. Experience has shown that in the latitude of Philadelphia it is not wise to allow any drainage to be exposed to the air in winter on account of freezing. The down-spouts should be continuous, and either run through the abutment into a French drain back of it, or be taken to the ground level outside, or preferably into a sewer without a trap. During the past 6 years there has been no case where freezing has given trouble in a closed pipe, but, with the detail referred to, the water has frozen on the bridge seat and in melting has run down the face of the abutment.

Allowing the drainage to go over the back wall is without doubt the most reasonable method, at first sight, but when consideration is given to the fact that the drainage of the roadbed is not ideal, and never is, nor can be, it is found that such a detail is not desirable. Even if the stone packing back of the abutments is placed as described by Mr. Auryansen, it will become clogged with dirt, will freeze under certain conditions, and the water will back up and force its way through the back-wall details to the bridge seat and down the face of the abutment. The writer's experience is decidedly in favor of removing it before it leaves the deck of the bridge.

The writer does not agree with Mr. Woodruff about the ability to care for any considerable quantity of water back of the back walls. No difficulty has ever been experienced in properly flashing around the down-spouts, if reasonable care, the proper low melting point, and ductile asphalt are used. If low-melting-point asphalt is used in place

of the flashing-angles along the web of a girder, and if it should deteriorate in any way, it can be readily replaced without interfering with traffic. Recent data from manufacturers of asphalt indicate that the most durable mixtures are generally those with low melting points. Time alone will tell how long any bituminous material will last under definite exposures. Mr. Wagner.

All down-spouts should be as short and as nearly vertical as possible. Horizontal or nearly horizontal gutters should be avoided. They will always give more or less trouble.

Mr. Jones refers to the advisability of carrying the drain-pipes into the abutment to prevent freezing. This point is well taken, and it is believed that all drain-pipes should lead untrapped to a sewer or pass through the abutment into a French drain back of it. Water falling on top of a bridge, or ice or snow melting and dropping down the open end of a drain-pipe, will freeze, under certain conditions, and the detail as shown in the figure is undesirable.

Mr. Carpenter calls attention to the fact that the drainage inlets should not be placed directly under the tracks. Mr. Himes suggests the use of at least two flanges encircling the inlets of the down-spouts. Although water will go around sharp corners, it does not like to do so, and their addition is a further precaution. Both these changes have been made in the specifications.

The desirability of a half-round cast-iron pipe to assist drainage is not altogether proved, but it is believed that it will facilitate its movement through the ballast on the deck of a bridge in the same way that it assists the water on the floor of a sand filter in reaching the main collectors.

Mr. Carpenter's suggestion, to caulk the edges of the steel at joints in the steelwork, does not appeal to the writer, although, at times, when a bad leak has occurred at a critical place, he has wished that it had been done. Generally, however, such a method, besides being very expensive, does nothing whatever to protect the steelwork from corrosion.

The suggestion of Mr. Jones, to use cement mortar instead of burlap dipped in hot asphalt, is excellent, and Paragraph 12 has been modified to allow of its use. The last sentence of Paragraph 46 has been cut out, for the reason given by Mr. Jones. The question as to the character of the protection has been referred to in another part of this discussion.

Mr. Vail's method of protecting the joint at the ends of abutting girders is a good one for a difficult location. It is rather complex, but seems to do the work well.

It is believed that it is unnecessary to use a 1:2:4 mixture for the concrete filler. The ½-in. coating of cement mortar is placed immediately after the concrete is laid and is incorporated with it in

Mr. Wagner.

the same manner as the granolithic finish of a cement pavement is applied; in fact, Mr. Jones advocated the use of a granolithic mixture, which again hardly seems necessary.

The specifications used by the writer for the steelwork of bridges of this type provide that no paint shall be applied where either concrete or water-proofing come in contact with the metal.

The use of brick as a protection to water-proofing has caused considerable comment. Brick laid in asphalt with the joints filled with the same has advocates, but it is not recommended, as it may move and tear the membrane. It is also difficult to get really good adhesion under ordinary working conditions. On the other hand, when it is laid in cement mortar and grouted, it is rigid enough to be satisfactory, and makes a solid mass, which, however, can be removed in sections without much trouble, in case repairs are necessary. This refers to its application on practically horizontal surfaces. It is believed that the protection of haunches along girders can be done best with reinforced concrete, and, if preferred, the same method may be used over horizontal surfaces. From the best information obtainable, the cost of brick in cement will be about the same as 2.5 in. of reinforced concrete, unless a high grade of reinforcement is used, in which case the cost will be slightly greater. It has been found difficult to lay brick in asphalt compound as protection on haunches, on account of the movement caused by the large number of plastic joints.

3.—Materials.

(*a*) *Felts and Fabrics.*—Mr. DeKnight raises a question that requires much additional light before it can be definitely settled. If felt or fabric is to be used, how many layers would be required? Practice differs very much in this respect, and the writer's experience has not yet shown what the solution should be. At the present time, this question has been studied by trying different methods, running all the way from five layers of felt to one layer of fabric, and thus far there is nothing to show which is the best. The structures are being carefully watched. Unfortunately, the discussion has not thrown any light on this important question.

It is true that experience with asbestos felt has shown that it is a very tender material and requires the utmost care in its application. It, however, contains such a large proportion of inorganic material as to make its use attractive when the question of durability is considered. Mr. DeKnight's data as to its history in the New York subways are interesting and deserve the attention of engineers who are considering its use.

Mr. Rhett asks the question: "If a stratum of material is truly and inherently water-proof, how can it be made more so by multiplying it?"

The writer believes that, theoretically, one thickness of a water-proofing material is sufficient if it is perfect and properly protected. His experience, however, shows that practically this is almost impossible, on account of the personal equation of the workmen, and that there is no factor of safety in such work. He believes, therefore, that it is good and safe practice to make more than one layer, certainly two, and possibly three, as a minimum. Mr. Wagner.

The writer generally agrees with Mr. Gardiner in the fact that flexibility is desirable in the felt or fabric used for a membrane. Not only is it easier to fit around stiffeners, gussets, etc., with a flexible felt or fabric, but it is easier to insure proper contact between its individual layers. This is particularly true when the work must be done in cold weather, when the compound itself is more or less viscous, and the contact must be made quickly to secure proper adhesion. As to the criticisms of the requirement which provides that the first layer shall be cemented to the concrete, it is believed that this method, all things considered, is the best. If the concrete cracks, and the membrane is tightly cemented to it, the strain on the membrane is localized and may cause rupture, whereas, if it is loose, the probabilities are that the stresses in the membrane will be distributed over an appreciable area, and there will be sufficient yielding to prevent rupture. As to the necessity of painting with asphalt the surfaces with which the flashing asphalt comes in contact, it is believed that good adhesion cannot be obtained without painting, and that the thin coating of asphalt secured by painting provides a double insurance. The additional cost is hardly worth considering.

The discussion of Mr. Walker, with respect to felts and fabrics, is very interesting, especially the fact that, if it is possible to obtain a fabric which is thoroughly saturated with asphalt, it is decidedly preferable to one in which the material is simply coated. The writer's experience shows that this matter is very important, and that felts have rotted where water has been able to reach their edges and penetrate into the center. There are many felts on the market on which practically all the asphalt is applied superficially.

Mr. Carpenter prefers to specify reinforced felt or treated cotton fabric in place of treated burlap. Since the paper was prepared, information has been received which inclines the writer to adopt this suggestion and accordingly he has changed Paragraph 9.

A number of long-time tests on fabrics and felts suspended partly in water and partly in the air shows in a general way that there is no real difference between coal-tar pitch and asphalt, as far as durability is concerned. These tests covered intervals of between one and two years, and seem to bear out the statement made by Mr. Walker.

(*b*) *Asphalts.*—Mr. Himes asks whether the tests described in Appendices A and B would insure a satisfactory quality of asphalt

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with the same certainty that one feels in the testing of cement or steel. The writer in reply would say that to him this is one of the most interesting points in the study of water-proofing materials, and that the tests given on pages 319, 320, and 324 were made, not only on samples selected from materials delivered for work under contract, but also on samples submitted by manufacturers for examination. It was the intention to take samples from each shipment on the contract work, as is done with cement, in order to determine the uniformity of the material. In all these tests the largest percentages were from materials on which no definite specification requirements were given, and, therefore, it would not be fair to answer the question from these tests alone. Again, the tests represent in many cases materials which were not used in the work, but which, in spite of the absence of specifications, were replaced by the contractor if not considered satisfactory. It is believed, however, that asphalts tested in the manner specified will enable one to determine, to a fairly close degree, whether he is obtaining a material of the quality required. The exact quality of a satisfactory asphalt opens a very wide field, in regard to which the writer feels only competent to say that, as far as he has gone, the requirements given in the specifications are all that he feels sure of at the present time. Any requirements which could be given to protect the consumer from the acceptance of materials which might contain substances which would affect the quality or durability of the asphalt would be welcome, such tests, for instance, as the requirements for sulphur and phosphorus in the specifications for steel. The writer has not at the present time the confidence in the technique of making the required tests on asphalt that he has in the accepted methods of testing cement or steel. In reference to this matter, it is interesting to note that the details of making the tests on bituminous materials referred to in the Appendices (which were the best known at the time the paper was written), have since been presented to the Society in a very complete form as the recommendations of the Special Committee on Materials for Road Construction, published in *Proceedings* for December, 1914 (the same number that contained the writer's paper), and, for this reason, he believes that Paragraph 36 should be changed to read as follows:

"36.—All tests shall be conducted according to the methods recommended by the Special Committee of the American Society of Civil Engineers on Materials for Road Construction, or such other methods as may be approved by the Chief Engineer."

It is of interest to note that in making these changes no essential difference is made in the practice of making the tests, but the recommendations of the Special Committee of this Society are more explicit in several cases than those of the American Society for Testing Materials. There can be no doubt that uniformity in the manner of

making the tests is of the utmost importance in comparing the qualities of different materials, and especially those of a bituminous nature. Mr. Wagner.

Mr. Walker has suggested that it would be wise to specify the temperatures to which the asphalt should be heated when applying it on the work. This suggestion would seem to be a wise precaution, and, therefore, Paragraph 40 has been modified so as to add the following:

"Asphalt shall not be heated at any time to a temperature higher than 450° Fahr. The temperature of the asphalt in the pail just before being mopped into the felt or fabric shall not be below 380° Fahr."

The idea of using an asphalt heater or some similar appliance for heating the concrete and driving out the moisture is to be commended. In fact, any method that can be used, when the work is not perfectly dry and the water-proofing must be continued should be seized with avidity.

The material used by Mr. Rhett is interesting, and it would be of great value to know how its properties differ from those specified by the writer for the elastic joint along girders. The writer, however, questions the advisability of attempting to use any material for this flashing without thoroughly cleaning the metal and the concrete. It does not seem wise to take any chances with a joint which is so important and can be made in a workmanlike manner with very little additional cost. The writer believes that in any bridge water-proofing the presence of an experienced inspector who shall be over the work as each part is done is a positive necessity.

The tests made by Mr. Goldbeck on the adhesiveness of the materials used in water-proofing are of great interest, and bear out the point found in practice that high ductility and high adhesive properties go together. Experience has shown that the point made by Mr. Goldbeck, about the less adhesion to concrete as compared with steel, is true in practice.

In Paragraph 41, a ductility of "at least 3 cm." is specified for the low-melting-point asphalt, and as it would seem that such a requirement would not be out of place in Paragraph 35, it has been added in the revised specifications.

Mr. Babcock asks the reason for giving the tests in Table 1. How these were brought about has been explained. The asphalts in this table, which comply with the requirements given in the specifications (Paragraphs 31 to 36, inclusive), have given good results up to the present time.

(c) *Coal-Tar Pitch.*—Answering Mr. Quimby's question: the writer has used coal-tar pitch, but never for solid-floor steel bridges, because he has been fearful of that property which makes it so much more brittle than asphalt in cold weather and so soft in hot weather that it is hard to control. He has been unable to obtain any of this material which has any measurable ductility at 40° Fahr., although

Mr. Wagner. there is no difficulty in obtaining an asphalt which has ductility at even lower temperatures. When the compound is used in such a manner that both ductility and adhesion are required, and when there is considerable vibration in the structure, which will occur in cold weather, it is believed that coal-tar pitch is not the material indicated for such use. On the other hand, the writer has used it successfully in structures free from vibration where ductility and adhesion are not specially required.

Where there has been no vibration, as in the case of a concrete arch, he has used coal-tar pitch, with the following specification, and, at the present time, the work is in good condition:

Coal-Tar Pitch.—The pitch shall be straight-run residue obtained from the distillation of coal-tar, and shall meet the following requirements:

1.—Melting point, not lower than 120° Fahr., nor higher than 140° Fahr.

2.—Matter insoluble in benzol, not less than 15% nor more than 35 per cent.

3.—Specific gravity at 60° Fahr., not less than 1.24 nor more than 1.34.

4.—Evaporation loss, 7 hours, at 325° Fahr., not more than 12% for pitch of 130° to 140° melting point; and not more than 14% for pitch of 120° to 130° melting point.

5.—Specific gravity of distillate at 670° Fahr. not less than 1.06, determined at 140° Fahr., compared with water at 140° Fahr.

As to the relative durability of asphalt and coal-tar pitch, referred to by Mr. Carpenter, the writer wishes that there was more definite information on the subject. Certain well-defined kinds of asphalts have been known to have stood up under years of service, but the coal-tar industry is so complex, and practice in the distillation of the pitch and the use of its constituents has varied so greatly that the writer has felt rather uncertain as to whether the pitch which was used many years ago and has stood so well is the same as that furnished to-day.

(*d*) *Asphalt Mastic.*—At the time the paper was prepared, rock asphalt mastic was not looked on as the most desirable material for water-proofing this type of bridge by the Masonry Committee of the American Railway Engineering Association, in its report on water-proofing, and the writer was inclined to feel the same way, to some extent. Since that time several cases have come under observation where the mastic has been found to be in remarkable condition after 20 years' service, and several large structures finished with this material, with special attention paid to details which were found defective in the past, have been showing up very well through two to three winters. It is believed that much of the uncertain feeling which the writer had about this material was unjust, and that the trouble was not with the material, but with the design and application.

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Data recently collated show that the proportions given in the specifications are as nearly right as can be expected to give good results in a climate similar to that of Philadelphia, and this mix when laid in two layers totaling 1.5 in. in thickness will be of such a consistency as to prevent cracking in cold weather and also injury from ballast when placed directly on top of it without any protection. A slightly richer mixture is required on sloping surfaces, and experience has shown that a good mastic worker can readily determine what this should be, in order to give the desired results; that is, if he can get it on satisfactorily, there will be no trouble afterward.

The question raised by Mr. Allen as to the proper identification of rock asphalt mastic is a good one, and his wording has been incorporated in Paragraph 50. The writer agrees entirely with all that he says, and after using this material for some years realizes the great danger of having a substitute used on the work, and the absolute impossibility of guarding against the use of other materials except in the manner proposed. It was intended that the first sentence of Paragraph 52 would cover this, but Mr. Allen's suggestion is an improvement.

The question raised by Mr. Lawrence, as to the selection of the best type of water-proofing, is very pertinent, although it was the intention to leave that question to the taste and fancy of the designer, in the absence of definite knowledge as to which was the best. As far as present experience is concerned, it is believed that if asphalt mastic, made of natural rock asphalt, properly mixed and applied, is used, there is very much to be said in its favor on the score of durability. The bitumen is the only organic material in it, and that, if properly mixed, is not likely to suffer injury, whereas it is known that felt or fabric will be acted on and decay. It is believed that some type of joint, as suggested by Mr. Lawrence, is advisable under certain conditions, say, at points where it is expected that cracking will occur, but the writer would reduce the width of such a joint to not more than 1 in. Experience with a large quantity of mastic, directly under the action of the ballast, without any protection, has shown that no cases have ever occurred where the ballast has cut into the mastic sufficiently to cause trouble. It is important that all contacts with metal, around stiffeners, drainage pipes, etc., should be made with pockets of low-melting-point asphalt, as the mastic will not adhere on account of the low bitumen content.

The following more detailed specification will be used:

"In applying the mastic adjacent to expansion joints, and around inlets, drain-pipes, etc., care shall be taken to insure adhesion of the mastic to the abutting surfaces, and, wherever directed, the joints shall be made with pockets of low-melting-point asphalt as provided in Paragraph 53."

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REVISED SPECIFICATIONS FOR WATER-PROOFING SOLID-FLOOR RAILROAD BRIDGES.

Part I. Details of Construction.

1.—Depth.—The depth of steel or concrete construction shall be such as to allow a sufficient distance from top of rail to top of steel or concrete floor for proper water-proofing and protection from the cutting action of the ballast. Under ordinary conditions, a depth of from 3.5 to 4.0 ft. from top of rail to clearance line below is sufficient.

2.—Drainage Grade.—Provision shall be made for grades of at least 1% on the floor of the bridge, to remove water promptly. Where this cannot be done in the steelwork, cement concrete, with a minimum thickness of 2½ in., shall be placed so as to drain the water to the inlets.

3.—Inlets.—Cast-iron inlets, with flanges embedded in the concrete shall be set at points not beneath the tracks where shown on the plans, and shall be provided with movable top grates. The down-spout from each inlet shall be provided with a trap and clean-out, which shall be accessible from below the bridge. The down-spout shall be of wrought iron, and connected to a sewer or arranged according to local conditions.

4.—Details of Steelwork.—Where two longitudinal girders meet over a column, the end stiffeners shall be placed with the outstanding leg toward the center of the girder, and the girders shall be connected with ⅜-in. plates on each side of the web. A plate shall also be placed covering the joint on the top flange.

5.—Stiffening angles on the outstanding edges of gusset-plates in half-through or through girders shall be terminated below the level of the water-proofing, the gusset-plates shall be heavier than usually designed when stiffening angles extend the full length of the plate, and shall be placed from 10 to 12 ft. apart.

6.—Where the top of girder approximates the same height as top of rail, the water-proofing and protection shall cover the entire top of the girder.

7.—The apron-plate from the steel floor over the back wall shall be provided with a curb angle against which to finish the water-proofing, and to this angle shall be riveted a vertical plate to prevent dirt from collecting under the apron-plate. The apron-plate shall be anchored to the back wall.

8.—Water-Proofing.—General Design.—On top of the prepared surface of the concrete shall be placed either of the following:

1st.—One or more thicknesses of felt or fabric, of quality and applied as specified hereafter, together with proper protection;

2d.—Asphalt mastic at least 1½ in. in thickness, of quality and applied as specified hereafter.

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9.—Felt or Fabric.—When water-proofing of this kind is to be used, either of the following types shall be adopted:

1st.—From four to six layers of felt;

2d.—One middle layer of treated cotton fabric, with four layers of felt;

3d.—One layer of felt, two layers of treated cotton fabric, and two layers of felt;

4th.—One middle layer of treated cotton fabric, and two layers of asbestos felt;

5th.—Either one, two, or three layers of treated cotton fabric.

10.—Protection of Water-Proofing.—After the completion of the felt or fabric water-proofing, the entire surface shall be covered and protected by one of the following methods:

1st.—Straight, hard-burned brick laid flat, with joints filled either with water-proofing compound or cement grout; water-proofing compound should only be used as a filler on flat or nearly flat surfaces;

2d.—A layer of cement concrete, from 2 to 2½ in. thick, with wire reinforcement;

3d.—A layer of about 1½ in. of asphalt mastic used only on top of asbestos felt. It is believed that no protection is necessary for the asphalt mastic.

11.—Special Drainage over Protection.—On top of the protection coat, and outside the line of the ties, a line of half-round cast-iron pipe, 6 in. in diameter, and perforated frequently, shall be placed to collect the water and convey it to the inlets.

PART II. PREPARATION FOR WATER-PROOFING.

12.—Preparation of the Steel.—All openings in the steelwork shall be closed, either by caulking with burlap dipped in hot asphalt, by the use of cement mortar, or by the use of sheet metal sufficient to retain the concrete base before applying the burlap and mastic.

13.—Preparation for Water-Proofing Materials.—Wherever called for by the plans, the decks of the bridges shall be protected with 1 : 3 : 5 concrete, with ¾-in. stone or gravel, mixed as specified hereafter, finished with a 1 : 2 mix of cement mortar, ½ in. thick, troweled to a smooth surface on top, as shown. This concrete shall be allowed to dry thoroughly so as to prevent the formation of steam when the hot water-proofing materials are applied.

14.—All vertical or sloping surfaces of concrete or steel shall be thoroughly cleaned of dust, dirt, loose particles, paint, and grease. The use of hand-bellows is recommended for cleaning loose dust and dirt from the surfaces. For cleaning paint and grease from the steel,

Mr. Wagner. and freshening the surfaces of asphalt, where a junction of old and new is to be made, or where a pocket of pure asphalt is used against the girders and the felt or mastic, gasoline shall be used, either by swabbing the surface with it, or by pouring a small quantity over the surface to be cleaned and setting fire to it. The use of a blow-lamp is also recommended.

15.—These surfaces shall then be painted with two coats of approved asphalt, diluted with gasoline. The materials of the first coat shall be proportioned so as to give a brownish tint. The second coat shall have a larger quantity of asphalt.

16.—Both coats of paint shall be thoroughly applied and worked into the surfaces, so as to give a uniform coating of the asphalt.

17.—Paint shall not be applied to damp concrete or steel. The painting shall be done immediately in advance of the application of the water-proofing materials and before dust has had time to collect.

18.—If the concrete is damp before the water-proofing is applied, the surface shall be first covered with a 2-in. layer of hot sand and allowed to stand for from 1 to 2 hours, after which the sand shall be swept back, uncovering sufficient surface to begin work, and the operation repeated over a new surface.

19.—Concrete Proportions.—The cement concrete shall be proportioned by measurement of volumes. The volume of a barrel of cement, 376 lb., shall be assumed to be 3.6 cu. ft. The sand and stone shall not be packed more closely than by throwing, in the usual way, into a barrel or box at the time of measurement.

20.—Cement.—Portland cement shall be used, of the quality specified by the American Society for Testing Materials, and all tests shall be conducted by the methods recommended by the Special Committee on Uniform Tests of Cement of the American Society of Civil Engineers, of January 17th, 1912.

21.—Stone.—The stone shall be clean, hard, crushed stone, or pebbles, to be approved by the Chief Engineer, and shall be composed of the whole run of the crusher, from ¼ in. to ¾ in. in size, screened of dust and particles less than ¼ in. in greatest dimension.

22.—Sand.—The sand shall be clean and sharp, and shall be composed of grains graded from "fine to coarse", screened to reject all particles of a greater diameter than ¼ in. It shall be free from foreign matter, and subject to the approval of the Chief Engineer.

23.—Care of Sand and Stone.—Sand and stone, when delivered on the work, shall be dumped on platforms, and not on the ground.

24.—Hand Mixing.—When mixed by hand, the cement and sand shall be first mixed dry and made into a mortar. The stone shall be spread on a suitable floor to a depth of about 6 in., thoroughly wetted, and the mortar evenly spread over it, care being taken that the stone

of each batch is mixed as to size. The whole mass shall then be turned over four times and raked, to secure complete and uniform mixture. If the Contractor desires to use some other method, he shall submit it for approval. Should the mixture be permitted to set before placing or tamping, it shall be removed and not used. Hand-mixed batches shall not be larger than 1 cu. yd. in volume. **Mr. Wagner.**

25.—Depositing.—All concrete shall be deposited as the Chief Engineer shall direct. It shall be of such consistency that when dumped in place it shall not require much tamping, and shall be laid with a view to be an aid to the water-tightness of the structure, and not merely a support for the water-proofing materials. All showing surfaces shall be troweled to a smooth, hard surface.

26.—In cases where concrete haunching against girders is called for by the plans, forms shall be used, and the concrete shall be of a wet consistency.

Part III. Materials and Application.

Water-Proofing Felt or Fabric and Asphalt.

27.—Materials.—On the prepared surface, apply the specified number of layers of approved saturated and coated felt, with a finished surface, and weighing about 14 lb. per 100 sq. ft.

28.—The bids shall be based on the use of the type of felt specified in Paragraph 27, but additional alternate bids will be considered, based on felts or fabrics other than these, which shall be approved by the Chief Engineer. In the event of such alternate bids being made, the bidders shall present with them sufficient data as to the method of manufacture, quality of materials, and references to places where such felts or fabrics have been used, giving dates of application.

29.—All materials shall be delivered on the work in their original packages, and properly branded.

30.—The acceptance or rejection of an asphalt shall rest with the Chief Engineer, and shall be based on the requirements stated in Paragraphs 31 to 37.

31.—The asphalt used shall consist of fluxed natural asphalt, or asphalt prepared by careful distillation of asphaltic petroleum.

32.—It shall contain, in its refined state, not less than 98% of bitumen soluble in cold carbon-disulphide. The remaining ingredients shall be such as not to exert an injurious effect on the work.

33.—When 20 grammes are heated for 5 hours at a temperature of 325° Fahr., in a tin box $2\frac{1}{2}$ in. in diameter, it shall not lose more than 2% by weight, nor shall the penetration at 77° Fahr., after such heating, be less than one-half of the original penetration.

34.—The melting point shall be between 150° and 190° Fahr.

35.—A briquette of the solid bitumen, having a cross-section of 1 sq. cm., shall show a ductility of at least 3 cm. at 40° Fahr., and at

Mr. Wagner. a temperature of 77° Fahr. shall show a ductility of not less than 20 cm., the material being elongated at the rate of 5 cm. per min. (Dow moulds).

36.—All tests shall be conducted according to the methods recommended by the Special Committee of the American Society of Civil Engineers on Materials for Road Construction, or such other methods as may be approved by the Chief Engineer.

37.—The penetration indicated herein refers to the depth of penetration, in hundredths of a centimeter, of a No. 2 cambric needle, weighted to 100 grammes, at 77° Fahr., acting for 5 sec.

38.—Application.—All flashing and reinforcing around inlets and other places specified shall be carefully executed.

39.—Water-proofing shall not be done in wet weather, or at a temperature below 32° Fahr., without special orders from the Chief Engineer. The felt shall be laid shingle fashion, the first two layers longitudinally and the last three transversely to the center line of the bridge, where five layers are called for, and as specified in detail in other cases, and shall be carried up the haunching and made secure against the girder in a satisfactory manner, or as shown on the plans. The flashing against vertical or inclined surfaces shall be in accordance with the directions of the Chief Engineer, if not indicated. The first layer of felt shall not be cemented to the floor of a steel bridge, except around the drain outlets. At no point shall there be less than the specified number of thicknesses.

40.—As the hot asphalt is spread, the felt shall be immediately rolled into it, and rubbed and pressed over its surface, so as to eliminate air bubbles and insure thorough sticking. One mopful of the asphalt shall not be spread over more than 1 sq. yd. of surface at one mopping. Not less than 2.5 to 3 gal. of asphalt shall be used on 100 sq. ft. of a single layer of felt. The top layer shall also be mopped and the work done so that the layers shall be one compact mass. Asphalt shall not be heated at any time to a temperature higher than 450° Fahr. The temperature of the asphalt in the pail just before being mopped into the felt or fabric shall not be below 380° Fahr.

41.—The finish of the water-proofing against the girders or concrete shall be made with a pocket of pure elastic asphalt of the quality specified in Paragraphs 31 to 37, except that the melting point shall be between 140° and 180° Fahr., the ductility at 40° Fahr. shall be at least 3 cm., and the adhesive qualities shall be satisfactory to the Chief Engineer. The surfaces with which this material comes in contact shall be dry, absolutely free from dust or grease, and, previous to its application, shall be covered with a thin paint made by dissolving the asphalt in gasoline, as specified in detail in Paragraph 15.

42.—Particular care shall be taken to make tight joints around gussets, stiffeners, and the ends of girders.

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43.—Care shall be taken to prevent injury in any way to the water-proofing by the passing of men and wheel-barrows over it, or by throwing any foreign materials on it.

44.—After the water-proofing course has been completed, the horizontal surfaces shall be protected, as shown on the plans, by a course of straight, hard-burned and dense brick, laid flat in a bed of 1 to 3 cement mortar, with full joints. There shall be not less than ½ in. of mortar between the felt and the bricks. The brick shall not increase in weight more than 10% when immersed in water for 7 hours.

45.—The haunching, and about 18 in. in width of the horizontal surface adjacent to the haunching, shall be protected, as shown on the plans, by about 2½ in. of 1:3:5 concrete, reinforced with No. 8 and No. 10 wire cloth, electrically welded, having a 3 by 8-in. mesh.

46.—Every care shall be taken to insure satisfactory and thoroughly water-tight joints between the main layers of water-proofing and the girders; and special attention shall be given to stiffeners, gussets, etc.

47.—Rolls of felt shall be stored on end, and not laid on their sides.

48.—Water-proofing shall be done only by experienced and expert felt water-proofers.

Natural Rock Asphalt Mastic.

49.—*Rock Asphalt Mastic.*—Wherever called for by the plans, the decks of bridges shall be water-proofed with natural rock asphalt mastic, as specified in Paragraph 50.

50.—The cement concrete, prepared as specified heretofore, shall be water-proofed with asphalt mastic equal in quality for the intended purpose, as to ingredients used and resistance to water, to the following specifications, and be approved as such:

Sicilian rock asphalt mastic......................60 parts.
Clean, sharp, graded grit and sand to pass a sieve of
8 meshes per inch..............................30 parts.
Asphalt as specified in Paragraphs 30 to 37.........10 parts.

These proportions shall be varied when required by special conditions on the work. The asphalt rock mastic blocks shall be imported European asphaltic rock mastic, brought to the place of work in original blocks with the brand stamped thereon. The mastic blocks shall be shipped to the site of the work on a through bill of lading.

51.—The mixture shall be made at the site of the work, shall be heated to a temperature of from 250 to 300° Fahr., and shall be stirred until all the ingredients are thoroughly incorporated. It shall then be spread and thoroughly worked, to free it from voids, and shall be ironed to a smooth surface with smoothing irons, if so directed. All mastic shall be applied in two coats, making the total thickness shown

Mr. Wagner. on the plans. The two coats shall break joints, and the mastic shall be distributed evenly. Where the thickness of the concrete plus mastic is less than 2½ in., the full thickness shall be made up of asphalt mastic.

52.—Water-proofing shall not be done in wet weather, or at a temperature below 32° Fahr., without special orders from the Chief Engineer.

53.—Pockets of asphalt shall be placed against all metal, and the mastic along girders, around stiffeners, gussets, etc., as specified in detail in Paragraphs 14 to 18, inclusive, and 41.

54.—In applying the mastic adjacent to expansion joints, and around inlets, drain-pipes, etc., care shall be taken to insure adhesion of the mastic to the abutting surfaces, and, wherever directed, the joints shall be made with pockets of low-melting-point asphalt, as provided in Paragraph 53.

55.—After the mastic is laid, it shall be mopped with pure melted asphalt, and the surface shall be spread with a layer of clean, coarse sand to harden the top.

56.—The pockets of asphalt placed against the girders, stiffeners, and gussets shall be protected, as shown on the plans, by about 2½ in. of 1:3:5 concrete, reinforced with No. 8 and No. 10 wire cloth, electrically welded, having a 3 by 8-in. mesh.

General Conditions.

57.—General Conditions.—The furnishing and erection of the steelwork for the bridge to be water-proofed will be executed under a separate contract, and the riveting will be completed, the erection finished, and the steel floor cleaned up ready for the water-proofing before the work on this contract is begun. In addition to the foregoing, the Contractor shall make a final cleaning of the steelwork before the work of water-proofing is begun.

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TRANSACTIONS

Paper No. 1339

RECONSTRUCTION OF THE NORFOLK AND WESTERN RAILWAY COMPANY'S BRIDGE OVER THE OHIO RIVER AT KENOVA, WEST VIRGINIA*

By William G. Grove,† Esq.,
and Henry Taylor, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. C. H. Cartlidge, T. Kennard Thomson, F. W. Skinner, L. L. Jewel, and William G. Grove and Henry Taylor.

Synopsis.

The purpose of this paper is to set forth in a general way, by using a specific case, the difficulties that engineers must surmount when it becomes necessary to replace an existing structure, which has become inadequate for the demands of modern service, with another structure capable of meeting present and future conditions.

In this instance, the problem consisted of replacing a single-track, five-span bridge, 1 800 ft. long and 100 ft. above water level, with a double-track bridge, on the same piers, without interruption either to railway traffic or river navigation.

The methods used in the reconstruction are described, as well as the design of the new structure. The paper is divided into eight parts, as follows:

History and description of the old bridge;

Erection of the viaduct approach;

*** Presented at the meeting of February 17th, 1915.**

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Various preliminary schemes for the erection of the river spans;

Description of the new spans, with loads and unit stresses used in the design;

Method of remodeling the tops of the present piers;

Description of the pier girders and material tracks;

Method finally adopted and used for the reconstruction of the five river spans;

Arrangement of the material yard.

The writers, as far as possible, have endeavored to make the paper a descriptive one, and hence have omitted all the calculations necessary in the design and in ascertaining the deflections of the several spans. The calculated figures, however, agreed very closely with the distances actually measured in the field.

By the addition of drawings and photographs, it is hoped to show the actual field conditions in a much clearer way than could be done by description. Figs. 46 and 47, showing the flood of March, 1913, tell better than words why the extra precautions were taken during this reconstruction.

By this method of description and illustration, the writers hope to make the paper interesting and instructive to those members of the Society who are not connected with the structural end of engineering, as well as to bridge engineers.

In conclusion, the point to be emphasized is the practicability, from an economic as well as from an engineering standpoint, of safely reconstructing existing structures without serious interruption of traffic.

Introduction.

The original single-track Norfolk and Western Railway bridge across the Ohio at Kenova, W. Va., having become inadequate for the heavy traffic of the road, it was decided in 1909 to replace it with a heavier double-track structure. As this bridge was on the only practical route for the Railway Company's traffic, the problem presented to bridge builders was one of replacing the bridge and at the same time not disturbing the traffic over it, or river navigation. As the tops of the piers were about 100 ft. above the water, and as the construction of new piers would have involved quite a considerable additional expense, the question ultimately resolved

itself into one of replacing the bridge with a new structure on the same piers.

The paper describes the manner in which this reconstruction, by the method of cantilever erection, was accomplished successfully by the American Bridge Company during 1912-1913, without any interference with river navigation and, except for one day, without interruption of railway traffic.

History and Description.

The Town of Kenova is on the south bank of the Ohio River, in West Virginia. It is just opposite the extreme southern part of Ohio, and is separated from that State by the Ohio River, and from Kentucky by the Big Sandy River which empties into the Ohio at this point, hence the derivation of the name: Ken for Kentucky, o for Ohio, and va for Virginia, and it is therefore called the Tri-State City.

At this point the Ohio River flows from east to west. On the north or Ohio shore the bank is very steep and the hills extend almost to the river bank. On the south shore, however, conditions are different. The hills begin to rise about ½ mile back from the river, and Kenova is on the plain between the hills and the river. The tracks of the Baltimore and Ohio Railroad and the Chesapeake and Ohio Railway run parallel to the Ohio River on the south bank. The Norfolk and Western Railway comes directly out of the hills on the south bank and, after passing along a viaduct over the Chesapeake and Ohio and the Baltimore and Ohio lines, crosses the Ohio River at right angles and at an elevation of about 100 ft. above mean water level. Trains cross the bridge at the rate of 72 per day, or about 1 every 20 min., and consist of from 50 to 75 cars each. These trains are made up principally of coal cars, bringing coal from the fields in the mountains of West Virginia and transporting it through Ohio, Indiana, etc.

The old bridge, Fig. 1, completed in 1891, was fabricated by the Edge Moor Bridge Company (now the Edge Moor Plant of the American Bridge Company), and was erected by Messrs. Baird Brothers. It consisted of five single-track, through, pin-connected truss spans, a channel span of 518 ft., flanked on each side by two 298-ft. spans. There was a 2 148-ft., single-track, deck, plate-girder, viaduct approach on the West Virginia side and one 64-ft., single-track, deck, plate-girder span on the Ohio side. The old bridge was designed for

Cooper's E-40 live load. The trusses were made 34 ft. from center to center, with the idea of putting a third truss in the center when it became necessary to double-track the bridge. All the spans were of the sub-panel type. The bottom chord was made up of two lines of eye-bars, 7 ft. 0 in. apart vertically; in the end panels, however, the two lines were brought together so as to connect to the single pin at the shoe. Floor-beams and stringers were placed between the two lines of eye-bars. In the intermediate panels, there were two sets of bottom laterals, one in the plane of the top flange of the floor-beams and the other in that of the bottom flange. In the end panels there was only one set of bottom laterals, which was in the plane of the shoe pins. All the top laterals, bottom laterals, and sway-bracing diagonals were of rods connected to the trusses by pins. A longitudinal strip of timber was placed on top of each line of stringers, and supported the wooden cross-ties, which, in turn, carried the rails.

At both ends of the bridge and viaduct the line had already been double-tracked, and, on account of the very heavy traffic and the increase in the weight of the loads carried, it became necessary to provide double track and, at the same time, increase the strength of the bridge. Various plans were considered, and the one finally adopted retained the same alignment as the old bridge, letting the new viaduct rest on the old footings and the new spans on the old piers.

Erection of Viaduct.

The new viaduct, designed in such a manner that it could be erected without interrupting traffic, was built in 1911. New footings had been placed under the old viaduct columns some years before. They were built on concrete piles, and each was designed to carry the load of a double-track viaduct column. The viaduct approach consisted of sixty-eight spans of 30 ft. 8¼ in., and one span of 61 ft. 4½ in., giving a total length of about 2 148 ft. New columns were provided throughout and new girders for all but twenty spans at the south end. For these twenty spans the old girders taken from the remainder of the viaduct were framed together in pairs, and two were used under each rail. These old girders were in good condition, and this substitution saved the expense of considerable new material. The viaduct runs through the sparsely settled part of the town, and its height varies from 20 ft. at the south end to 50 ft. at the north or river end.

FIG. 1.—THE OLD NORFOLK AND WESTERN RAILWAY BRIDGE AT KENOVA, W. VA.

FIG. 2.—THE NEW BRIDGE AT KENOVA, W. VA.

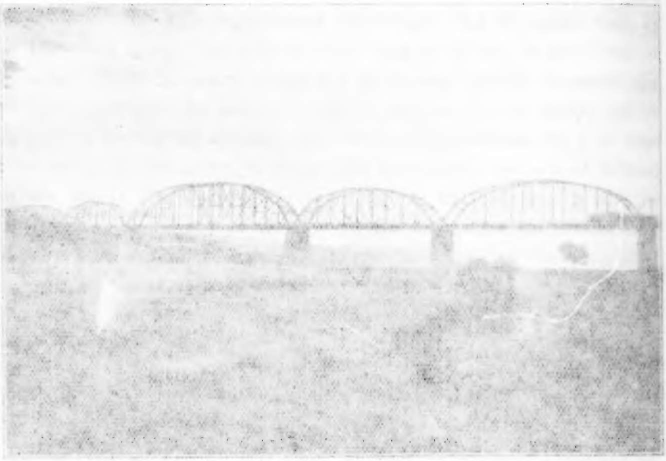


FIG. 1.—The Old Norfolk and Western Railway Bridge at Kenova, W. Va.

FIG. 2.—The New Bridge at Kenova, W. Va.

Erection was started at the north end and was carried on without interrupting traffic and without the use of falsework except under the first two spans. The traveler ran on rails laid over the new outside girders, and the first two spans had to be erected on falsework in order to have a place on which to erect the traveler. A track was laid on the ground alongside the viaduct for nearly its entire length, and the new material, as it was shipped in, was unloaded by a small derrick car near its proper place in the viaduct.

A derrick was placed on Pier 1, at the north end of the viaduct, and with it was erected the falsework and the first two new spans of the viaduct. The derrick then raised the traveler used in erecting the remainder of the viaduct. This traveler consisted of two wooden bents resting on wooden sills. The bents were shaped like those of an ordinary wooden gantry traveler, and were braced together. A platform, 21 ft. 6 in. above the rail, carried two hoisting engines which worked the booms on the forward end of the traveler. These booms were of steel, one 80 ft. and the other 65 ft. long. They turned on foot-blocks made fast to the front end of the sills, and each was capable of lifting about 12 tons. As there were no braces between the bents below 21 ft. 6 in. above the rail, trains were run through the traveler without interrupting its work in any way. The old viaduct being single-track, trains were carried on the two inside girders of the new work until the viaduct was completed. As a change of grade was found necessary in the new work, the Railway Company, some time before commencing the erection of the new steel, had jacked up all the old viaduct columns until the rail was at its proper level for the new work. Wood blocking was placed on top of the concrete pedestals to carry the old columns in their raised position. The new viaduct bents consisted of two columns connected by a cross-girder, on top of which rested four lines of longitudinal girders. The bents were connected by longitudinal bracing in alternate openings.

The method of erection (shown by Fig. 3) was as follows: The traveler erected two new bents on blocking level with the top of the concrete pedestals, one alongside each old bent. A temporary wooden strut was bolted to the old bents to brace them, and the old longitudinal bracing was removed. The ends of the old girders over these bents were fastened together and, using both booms of the traveler, the girders were hoisted a few inches, so as to clear the tops of the old

columns. This condition is shown by (1) of Fig. 3. The old bents were then pulled out to one side by lines from the traveler. New bents were pulled into place, and the old girders were slacked down on them, timber blocks on the tops of the columns being used to make up the difference in height of the old and new girders. Trains were then allowed to pass. The new longitudinal bracing was put in between the new bents, and the old bents were cut to pieces. This condition is shown by (2) of Fig. 3. The track was picked up at each end of the old girder span and blocked so as to clear the old

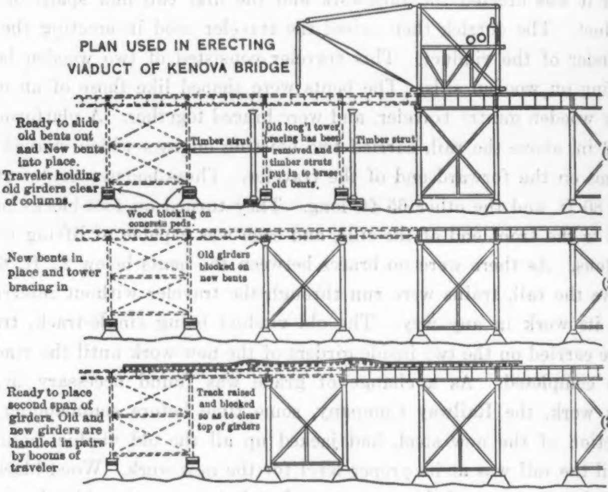


FIG. 3.

girders, the latter then being pulled out from under the track and laid on the ground below. The bracing between the girders was not cut out, so that, while one boom was removing the old girders, the other one hoisted the new girders into place. The new girders were shipped riveted up in pairs with bracing and cross-frames. The track was again picked up, the blocks taken out, the track slacked down, and trains allowed to pass. The second span of the old girders was handled in the same way. This is shown by (3) of Fig. 3. The traveler was then moved ahead two panels, and the operation was repeated.

None of these operations took more than 15 or 20 min., and it was very seldom that trains were held even a few minutes, as work

was not started if a train was seen to be approaching. The rate of progress was two spans, or about 60 ft. per day. The traffic, consisting mostly of coal trains, was very heavy during the reconstruction of the viaduct.

At the south end of the viaduct was the Kenova station, and this portion of the work was erected with a small derrick car, as the traveler would have interfered with the proper handling of passengers and trains. This portion of the old work was already double-tracked, and trains were allowed to run on one track while the derrick car erected the new work on the other. Traffic was then shifted, and the new steel for the second track was erected.

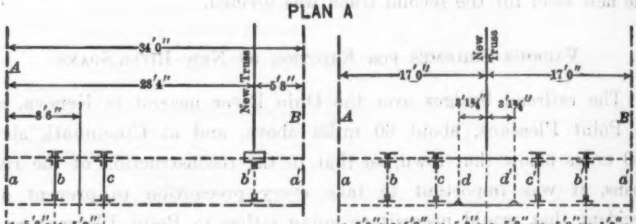
Various Schemes for Erection of New River Spans.

The railroad bridges over the Ohio River nearest to Kenova, are at Point Pleasant, about 60 miles above, and at Cincinnati, about 150 miles below that town, so that in the reconstruction of the river spans, it was important to take every precaution to prevent any accident that would necessitate going either to Point Pleasant or to Cincinnati to cross the river. Another condition which had to be considered was the stage of the Ohio River, the rise during floods amounting to as much as 70 ft. A third condition was the fact that provision had to be made for river navigation during the reconstruction. On account of these conditions and the very heavy traffic, the contract was awarded with the provision that traffic should be maintained at all times, and that no falsework should be used in the river.

Several designs for the river spans were worked up in the New York office of the American Bridge Company. The first design provided for the addition of a third center truss and the reinforcement of the two old outside trusses. New stringers were to be provided, but the old floor-beams were to be cut so as to use their ends and a new central portion which would connect to the new center truss. The floor-beam splice on each side of the new truss was to be designed so as to transmit shear but not moment, and by this means the live load transmitted to the outside trusses could be limited to a definite quantity by the location of the floor-beam splice, and thus not overstress the old trusses. Fig. 4 shows the details of this method.

The second design provided for placing falsework under the old stringers only and disconnecting the latter from the old floor-beams, so that the falsework would carry the old stringers, old track, and live load. New floor-beams were to be suspended from the old ones by rods, and new spans were to be built up around the old ones on brackets attached to the ends of the new floor-beams in suspended position. After the new spans were swung, the old ones were to be blocked up on them and then taken down. Finally, the new floor-beams were to be raised to their final position and new stringers added. Fig. 5 shows the details of this method.

PLAN A



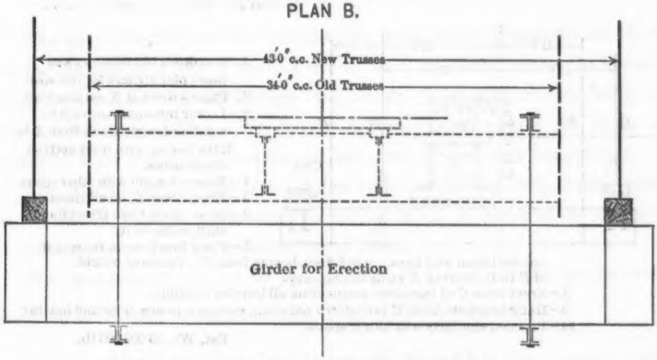
1.-Repair old trusses, rearrange old top laterals and portals to clear new truss when first erected, erect other new bracing.
2.-Erect new stringers at *b* and *c*.
3.-Remove old stringers and bolt one line to floor-beams at *b'*.
4.-Erect new truss at *b'*, swing it and remove old stringers.
5.-Move new truss to center line of bridge, connect it to transverse struts and erect new top laterals and portals.
6.-Block up stringers at any panel point on temporary erection girders hung from old trusses. Saw old floor-beam through at *d* and *d'* and remove old center section. Place new center section, connecting it to truss and to old end sections of floor-beam, making efficient shear connections and top flange tension splices, but making butt joints for compression only in the bottom flange.
7.-Add reinforcing plates to floor-beams from *c* to *d* and *c'* to *d'*.
8.-Erect new stringers at *b'* and *c'*.

FIG. 4. Est. Wt. 9 000 000 lb.

The third design eliminated the use of falsework entirely. New stringers were to be placed on top of the old floor-beams and a track was to be laid so that the base of rail would be about 8 ft. 9 in. above the present base of rail, and the center line of track about 10 ft. 0 in. from the original center line. Then the masonry piers under one span were to be cut down and the span was to be lowered and moved sidewise 5 ft. 9 in., the temporary track being shifted until it lined up with the track on the other spans. The other spans were then to be treated in the same manner. One new truss was to be erected in its permanent position on all spans and the truss swung; then the tempo-

rary track was to be shifted to its permanent location near the new truss. Brackets were to be attached to the ends of the old floor-beams, and one end of the latter was to be hung from the new truss just swung, so that the adjacent old truss would be clear of the masonry. The other new truss was then to be erected on the brackets in its permanent location and swung. The other ends of the old floor-beams were to be hung from the second new truss and the old spans taken down. Finally the new floor-beams and stringers were to be placed. Fig. 6 shows the details of this method.

PLAN B.



1. Cut down old piers and place new pier girders.

2. Support old stringers on falsework, cutting them loose from floor-beams, and carry traffic on falsework during erection of new trusses.

3. Erect new trusses outside of old trusses, as shown, connecting up bracing where possible.

4. Hang erection girders from new trusses and take down old span, connecting up new floor and bracing throughout.

Est. Wt. 16 000 000 lb.

FIG. 5.

Each design had certain objectionable features. The first gave a patched-up bridge. The second gave a new bridge, but required falsework under all the spans. The third design provided for a new bridge and eliminated the falsework, but the erection programme was very complicated, requiring the lowering and sliding of the old spans and considerable shifting of the track.

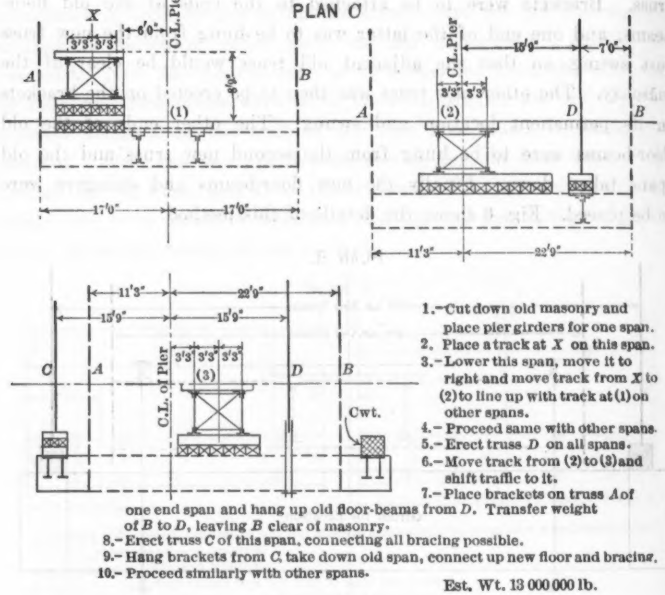

PLAN C
X
3'3" 3'3"
6'0"
C.L. Pier
A
B
(1)
17'0"
17'0"
C.L. Pier
15'9"
7'0"
3'3" 3'3"
(2)
D
B
A
11'3"
22'9"
11'3"
22'9"
15'9"
15'9"
C
A
3'3" 3'3" 3'3"
(3)
C.L. of Pier
D
B
Cwt.
1.–Cut down old masonry and place pier girders for one span.
2. Place a track at X on this span.
3.–Lower this span, move it to right and move track from X to (2) to line up with track at (1) on other spans.
4.– Proceed same with other spans.
5.–Erect truss D on all spans.
6.–Move track from (2) to (3) and shift traffic to it.
7.–Place brackets on truss A of one end span and hang up old floor-beams from D. Transfer weight of B to D, leaving B clear of masonry.
8.–Erect truss C of this span, connecting all bracing possible.
9.–Hang brackets from C, take down old span, connect up new floor and bracing.
10.–Proceed similarly with other spans.
Est. Wt. 13 000 000 lb.

FIG. 6.

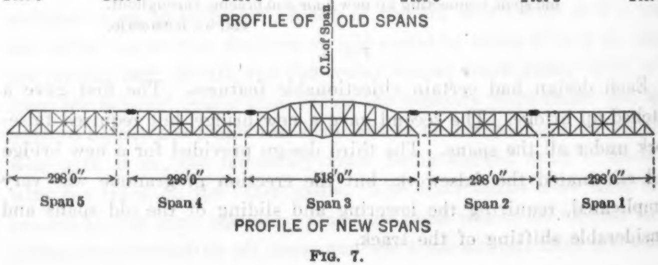

KENOVA BRIDGE
North
South
Ohio End
W.Va.End
298'0"
298'0"
518'0"
298'0"
298'0"
Span 5
Span 4
Span 3
Span 2
Span 1
Pier 6
Pier 5
Pier 4
Pier 3
Pier 2
Pier 1
PROFILE OF OLD SPANS
C.L. of Span
298'0"
298'0"
518'0"
298'0"
298'0"
Span 5
Span 4
Span 3
Span 2
Span 1
PROFILE OF NEW SPANS

FIG. 7.

These objectionable features led to the adoption of the final design which, in brief, was as follows (Fig. 7): Spans 1 and 5 were to be erected according to the method of the second design. Then Spans 2 and 4 were to be erected around these old spans by cantilevering out from Spans 1 and 5, respectively. Span 3 was then to be erected around the old span by cantilevering each end out from Spans 2 and 4, and joining the two halves of Span 3 in the center. Finally, the old spans were to be blocked up on the new ones and removed.

Description of New Spans.

The span lengths on the new bridge were made the same as those on the old one, but the trusses were 43 ft. 0 in. from center to center, so as to straddle the old bridge completely. All the trusses were of the riveted type. The four center bottom-chord panels of Span 3, however, were made of eye-bars on account of erection conditions.

The live loads for the new bridge on each of the two tracks were as shown by Fig. 8, and were placed in such a position as to give maximum stress in the member considered.

The dynamic effect due to the passage of trains on the bridge was provided for by the impact formula, $I = \dfrac{300}{300 + L}$, in which

$I =$ percentage of live load stress to be added for impact; and
$L =$ length of loaded track producing the live load stress.

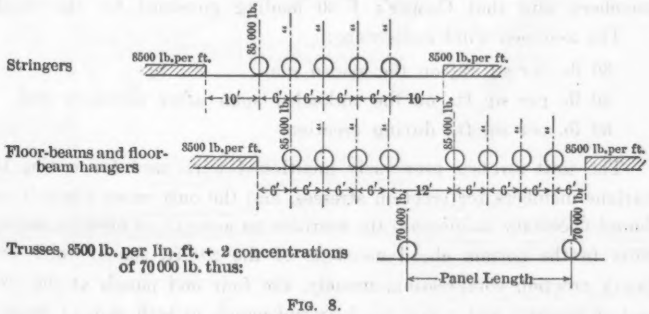


FIG. 8.

The structural steel for the new bridge had an ultimate tensile strength of from 62 000 to 70 000 lb. per sq. in., and the rivet steel an ultimate tensile strength of from 47 000 to 55 000 lb. per sq. in.

The allowable unit stresses were as follows:

Tension, 20 000 lb. per sq. in. (including dead, live, and impact stresses);

Tension, 24 000 lb. per sq. in. (including dead, live, impact, and wind and secondary stresses);

Tension in hangers, 16 000 lb. per sq. in.;

Compression, $\left(20\ 000 - 100\frac{l}{r}\right)$ lb. per sq. in.;

Extreme fiber stress on pins, 30 000 lb. per sq. in.;

Shear on shop rivets and pins, 15 000 lb. per sq. in.;

Shear on air-driven field rivets, 14 000 lb. per sq. in.;

Shear on hand-driven field rivets, 12 000 lb. per sq. in.;

Shear on plate-girder webs, 12 000 lb. per sq. in.;

Bearing on shop rivets and pins, 30 000 lb. per sq. in.;

Bearing on air-driven field rivets, 28 000 lb. per sq. in.;

Bearing on hand-driven field rivets, 24 000 lb. per sq. in.;

Bearing on masonry, 600 lb. per sq. in.

After the bridge had been designed according to these loads and unit stresses, it was tested for Cooper's E-60 live load on each track, using the American Railway Engineering and Maintenance of Way Association Specifications for 1910, and the design was modified accordingly where the latter conditions required the larger sections. It was generally found that the first condition governed for the web members, and that Cooper's E-60 loading governed for the chords.

The assumed wind loads were:

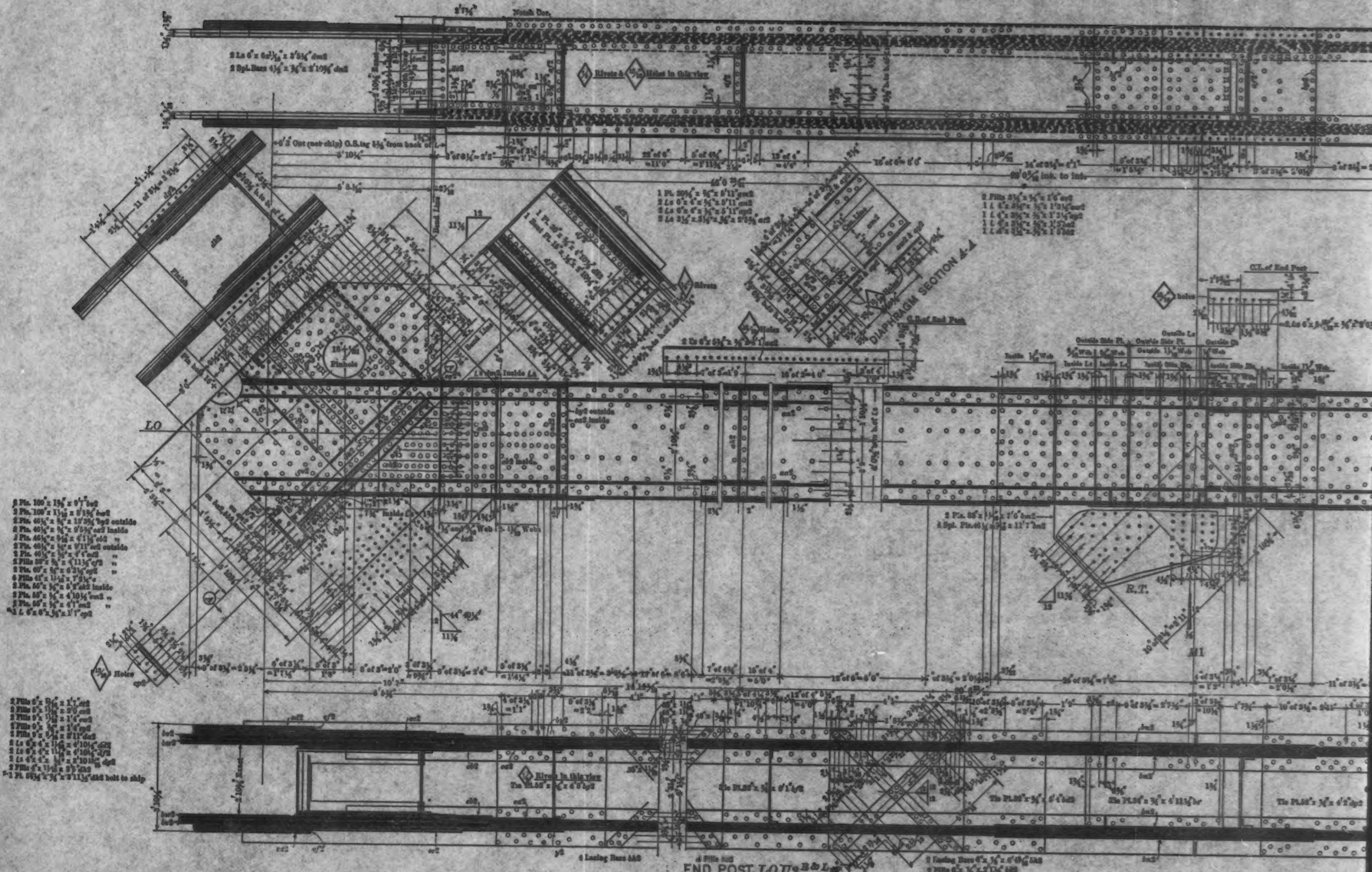
30 lb. per sq. ft., on the loaded span;

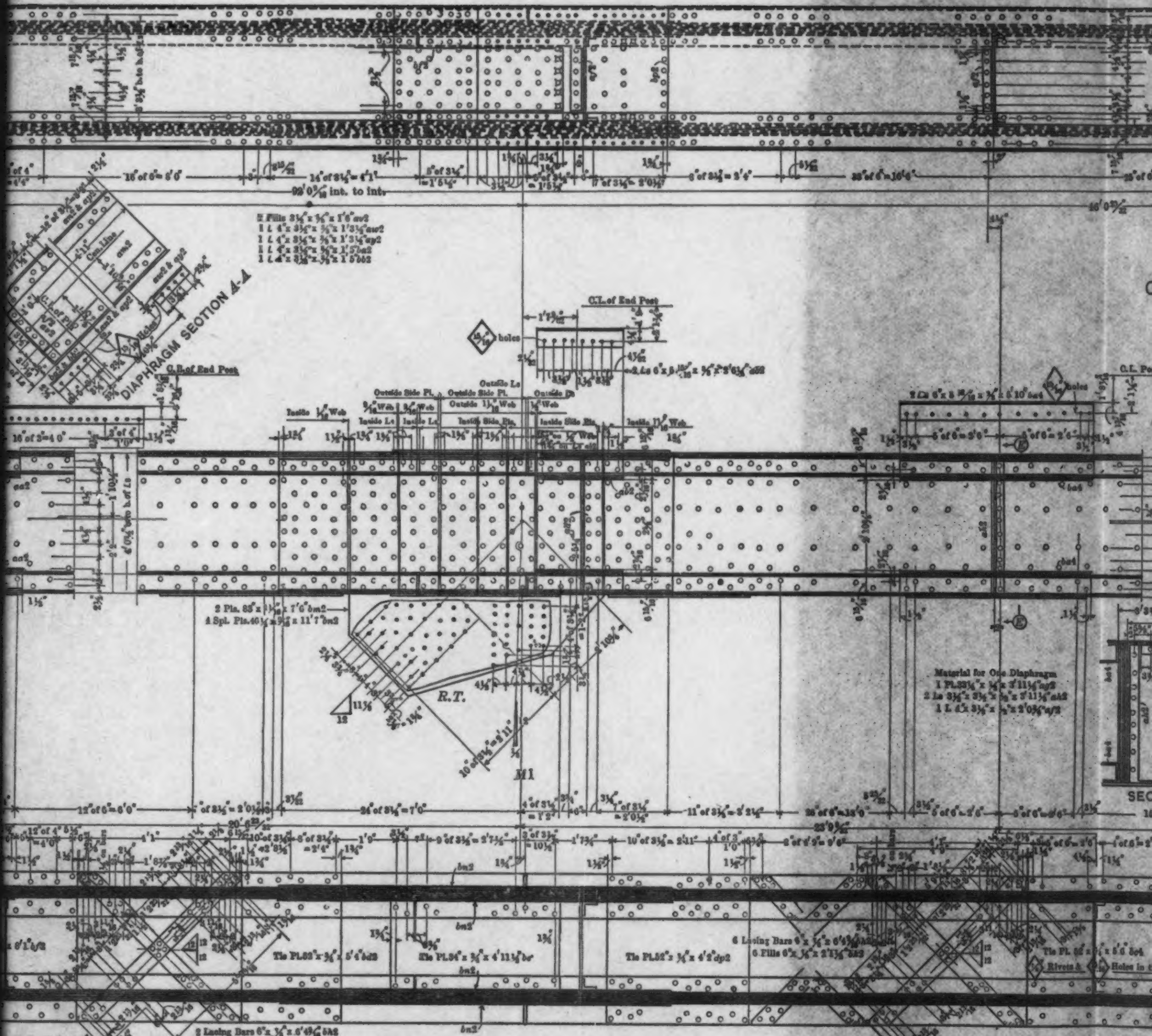
50 lb. per sq. ft., on the unloaded span after erection; and

50 lb. per sq. ft., during erection.

The unit stresses previously mentioned were used for testing the various members for erection stresses, and the only cases where it was found necessary to increase the sections on account of erection stresses were in the bottom chord members of the 298-ft. spans, which took heavy erection compression, namely, the four end panels at the river end of Spans 1 and 5 and the four end panels at both ends of Spans 2 and 4.

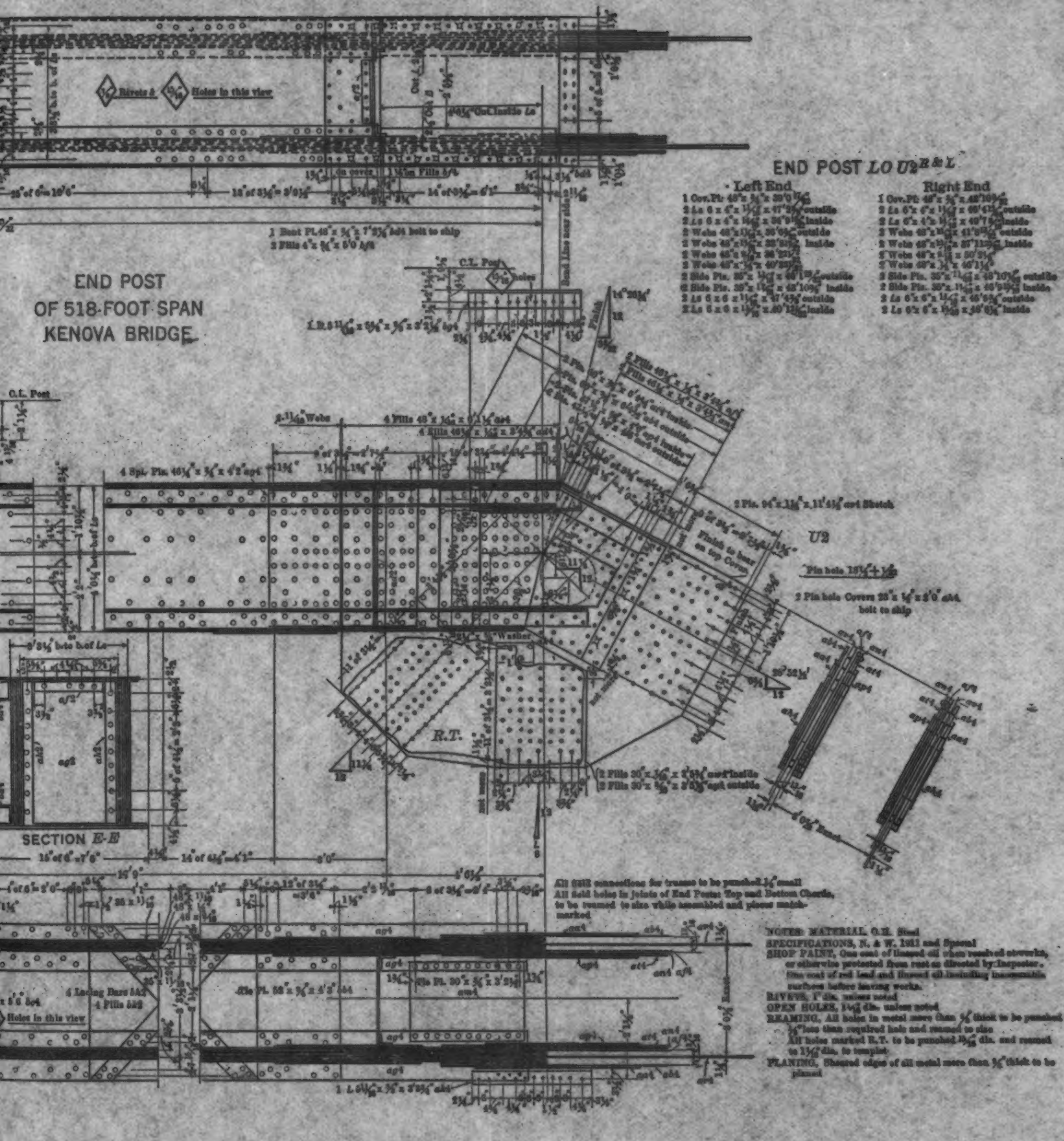
The rails on the new bridge are 100 lb. per yd. on the north-bound and 85 lb. per yd. on the south-bound tracks. The new ties are 8 by


DIAPHRAGM SECTION A-A
Pinhole
Rivets in this view
R.T.
LO
END POST LO U2



DIAPHRAGM SECTION A-A
C.L. of End Post
C.L. of End Post
holes
Outside Side Pl.
Inside Web
Outside Ls
Inside Side Pls.
R.T.
M1
Material for One Diaphragm
Tie Pl.

PLATE IX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXIX, No. 1339.
GROVE AND TAYLOR
ON RECONSTRUCTION OF KENOVA BRIDGE.



10 in., notched to $9\frac{1}{2}$ in. Each track was assumed to weigh 450 lb. per lin. ft. The assumed total dead loads per linear foot of bridge were 9 400 lb. for Spans 1, 2, 4, and 5, and 12 950 lb. for Span 3; and the actual total dead loads per linear foot of bridge were 9 380 lb. for Spans 1 and 5, 9 510 lb. for Spans 2 and 4, and 12 340 lb. for Span 3. All rivets are $\frac{7}{8}$ in. in diameter, except those in the trusses of Span 3, where 1-in. rivets are used. Fig. 9 is a diagram of the 298-ft. spans and Table 1 is the stress sheet. Fig. 10 is a diagram of the 518-ft. span and Table 2 is the stress sheet.

Special attention should be called to the fact that the trusses of the 518-ft. span are the largest riveted trusses ever built for a simple truss span.

Plate IX shows the details of the end post for the 518-ft. span. Fig. 11 shows a typical floor-beam of this span.

Remodeling the Tops of the Piers.

The length of the two piers under Span 3 was 46 ft. 0 in. under copings, and of the other four 43 ft. 0 in. As the new trusses were made 43 ft. 0 in. from center to center, so that they would clear the old spans, the shoe bearings of the new trusses came directly over the ends of the piers. It was necessary, therefore, to place distributing girders under the new trusses in order to transfer the reaction of these trusses in toward the center of each pier, and hence the piers had to be cut down to receive these pier girders. Two devices were used in order to reduce this pier cutting to a minimum. The rockers on the truss spans were made 36 in. deep, and narrow enough to be placed inside the bottom chord. Fig. 12 shows the position of the rockers of Span 3. Second, the pier girders consisted of three separate girders connected by diaphragms, and the cast-steel pedestals under the girders were made with ribs extending up between the girder sections and were connected to them by pins. In this way the pier cutting was reduced by about 6 ft., and it was only necessary to remove five courses of masonry, including the coping stones.

Fig. 13 shows the remodeled top of Pier 3. To form a neat finish to the tops of the piers after they were cut down, and also to give a strong bond, the Railway Company put a reinforced concrete coping around the three masonry courses immediately under the new steel. After it was determined just how many courses were to be removed,

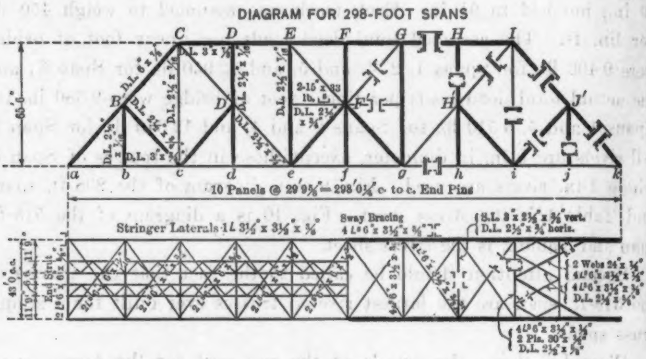


FIG. 9.

STRESSES, ETC., IN 298-FOOT SPANS.

END REACTION ON PIER.	END REACTION ON SPAN.		DEAD LOAD, IN POUNDS PER LINEAR FOOT.	
D = 797 000 lb.	D = 690 000 lb.	D = 690 000 lb.	Trusses 6 550	C. S. Pedestal 3 ft. 8 in. by 8 ft. 0 in.
L_1 = 1 404 000 "	L_1 = 1 260 000 "	L_2 = 939 000 "	Floor 1 950	100 lin. ft. of 36-in.
I_1 = 468 000 "	I_1 = 420 000 "	I_2 = 313 000 "	Track 900	rockers.
2 669 000 lb.	2 370 000 lb.	1 942 000 lb.	Total 9 400	
at 600 = 4 448 sq. in.				

PIER GIRDERS.

Shear.	*Moment.*	
D = 715 000 lb.	D = 42 740 000 ft-lb.	*Web.* 3 Pls. 46 × 5/8 = 86.3
L = 1 260 000 "	L = 75 600 000 " "	6 Fills. 34 × 15/16 = 165.8
I = 420 000 "	I = 25 200 000 " "	6 R. Pls. 44 × 5/8 = 165.0
2 395 000 lb.	143 540 000 ft-lb.	417.1 sq. in.
at 12 000 = 199.6 sq. in.	at 20 000 = 7 177 sq. in.	*Top Flange.* 6 Ls. 6 × 6 × 15/16 = 54.5g. = 44.7n.
		12 Covers 15 × 11/16 = 123.8g. = 90.8n.
		178.3g. = 135.5n.
		Bottom Flange. 6 Ls. 6 × 6 × 13/16 = 54.5 sq. in.
		9 Cov. 13 × 3/4 = 87.8 " "
		142.3 sq. in.

FLOOR-BEAMS.		STRINGERS.	
Shear.	*Moment.*	*Shear.*	*Moment.*
D = 43 400 lb.	D = 7 047 000 ft-lb.	D = 6 700 lb.	D = 599 000 ft-lb.
L = 343 000 "	L = 61 700 000 " "	L = 127 000 "	L = 9 800 000 " "
I = 246 000 "	I = 44 300 000 " "	I = 115 400 "	I = 8 920 000 " "
632 400 lb.	113 047 000 ft-lb.	249 100 lb.	19 319 000 ft-lb.
at 12 000 = 52 sq. in.	at 82.5 = 1 370 000	at 12 000 = 20.8 sq. in.	at 65.8 = 294 000
	at 20 000 = 68 5 sq. in.	*Web.* 69 × 1/2 = 34.5 sq. in.	at 20 000 = 14.7 sq. in.
Web. 82 × 11/16 = 56.4 sq. in.	1/8 web = 7.0 " "		1/8 web = 4.3 " "
2 Ls. 6 × 6 × 3/4 = 16.9g. = 13.9 sq. in. n.		2 Ls. 6 × 6 × 1/2 = 11.5g. = 10.5 " " n.	
5 Cov. Pls. 16 × 11/16 = 55.0g. = 46.0 " " n.			14.5 sq. in. n.
68.9 sq. in. n.		10 Int. stiffr. 4 × 3½ × 3/8 crimp.	

TABLE 1.—STRESS SHEET FOR 298-FOOT SPANS.

Member.	D.	L_1.	I_1.	$D + L_1 + I_1$.	L_2.	I_2.	$D \times L_2 \times I_2$.	W_1.	Sections for Spans 2 and 4.
a B'	− 838	− 1 701	− 569	− 3 108	− 1 269	428	− 2 532	8 610 000 ± 8.5	1 Cov. Pl. 38 × 13/16. 2 Ls. 6 × 4 × 3/4 4 Webs. 42 × 13/16. 2 Ls. 6 × 6 × 3/4 227.2 2 Pls. 29 × 1/2
B' C	− 745	− 1 511	− 506	− 2 762	− 1 099	− 368	− 2 212	8 610 000 ± 4.8	1 Cov. Pl. 38 × 13/16. 2 Ls. 6 × 4 × 3/4 4 Webs. 42 × 13/16. 2 Ls. 6 × 6 × 3/4 198.2
C D E	− 752	− 1 522	− 510	− 2 784	− 1 082	− 362	− 2 196	± 13 C D ± 32 D E	1 Cov. Pl. 38 × 13/16 = 30.9 2 Ls. 6 × 4 × 3/4 = 13.9
E F G	− 784	− 1 585	− 531	− 2 900	− 1 195	− 400	− 2 379	± 43	2 Ls. 6 × 6 × 3/4 = 16.9 4 Webs. 42 × 11/16 = 115.5 177.2
a b c	+ 564	+ 1 145	+ 383	+ 2 092	+ 854	+ 286	+ 1 704	± 41 a b ± 118 b c	4 Pls. 48 × 3/4 = 144.0 g. = 126.0 n.
c d e	+ 564	+ 1 164	+ 390	+ 2 118	+ 856	+ 287	+ 1 707	± 169 c d ± 206 d e	4 Ls. 6 × 6 × 3/4 = 33.8 g. = 27.8 n. 133.8 n.
e f g	+ 784	+ 1 585	+ 531	+ 2 900	+ 1 195	+ 400	+ 2 379	± 224	
C D'	+ 373	+ 874	+ 399	+ 1 646	+ 646	+ 295	+ 1 314	....	4 Pls. 30 × 5/8 = 75.0 g. = 65.0 n.
D' e	+ 280	+ 789 − 219	+ 360 − 137	+ 1 429 − 169	+ 605 − 168	+ 276 − 105	+ 1 161 − 86	....	4 Ls. 4 × 4 × 5/8 = 18.4 g. = 15.9 n. 80.9 n.
E F'	+ 47	+ 281 − 185	+ 141 − 103	+ 469 − 257	+ 217 − 145	+ 109 − 81	+ 373 − 195	....	2 Pls. 22 × 5/8 = 27.5 g. = 22.5 n. 4 Ls. 4 × 4 × 5/8 = 18.4 g. = 15.9 n. 38.4 n.
F' g	− 47	+ 185 − 281	+ 103 − 141	+ 257 − 469	+ 145 − 217	+ 81 − 109	+ 195 − 373	....	
C c	+ 227	+ 634	+ 354	+ 1 215	+ 452	+ 324	+ 1 003	....	4 Pls. 30 × 5/8 = 75.0 g. = 65.0 n. 4 Ls. 4 × 4 × 5/8 = 18.4 g. = 15.9 n. 80.9 n.
E e	− 86	+ 137 − 208	+ 76 − 104	+ 158 − 395	+ 107 − 161	+ 60 − 81	+ 112 − 325	....	2 Pls. 26 × 1/2 = 26.0 4 Ls. 4 × 4 × 1/2 = 15.0 41.0
Sub-Hanger	+ 89	+ 343	+ 246	+ 678	+ 258	+ 185	+ 532	....	2 Pls. 26 × 5/8 = 32.5 g. = 27.5 n. 4 Ls. 6 × 4 × 9/16 = 21.2 g. = 19.0 n. 46.5 n.
Sub-Diagonal	− 93	− 231	− 166	− 490	− 174	− 125	− 392	....	2 Pls. 20 × 9/16 = 22.5 4 Ls. 4 × 4 × 1/2 = 15.0 37.5
Sub-Post	− 48	0	0	− 48	0	0	− 48	....	2 Pls. 26 × 3/8 = 19.5 4 Ls. 4 × 4 × 3/8 = 11.4 30.9

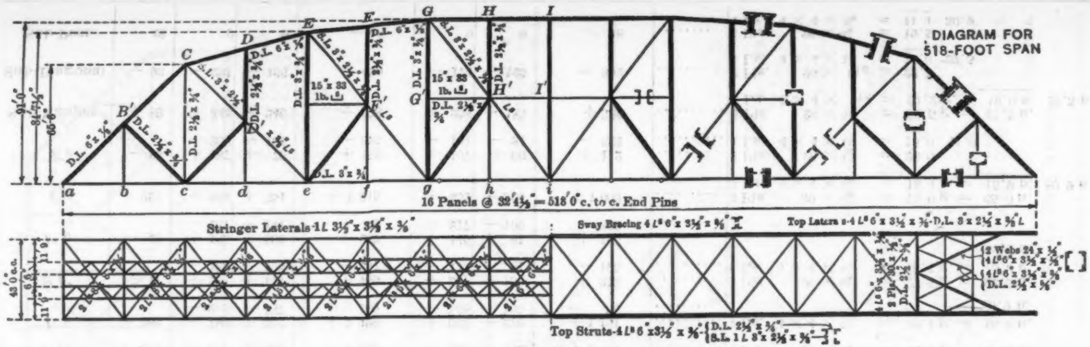


Fig. 10.

Stresses, Etc., in 518-Foot Span.

End Reaction on Pier.	End Reaction on Span.		Dead Load, in Pounds per Linear Foot.		Pier Girder.	
					Shear.	*Moment.*
D = 1 852 000 lb.	D = 1 712 000 lb.	D = 1 712 000 lb.	Trusses 10 050	C. S. Pedestal 6 ft. 0 in. by 9 ft. 0 in.	D = 1 740 000 lb.	D = 83 600 000 ft-lb.
L = 2 349 000 "	L_1 = 2 190 000 "	L_2 = 1 600 000 "	Floor 2 000	183 lin. ft. of 36-in. rockers.	L = 2 190 000 "	L = 105 000 000 " "
I_1 = 529 000 "	I_1 = 493 000 "	I_2 = 368 000 "	Track 900		I = 493 000 "	I = 23 600 000 " "
4 730 000 lb.	4 395 000 lb.	3 680 000 lb.	Total 12 950		4 423 000 lb.	212 200 000 ft-lb.
at 600 = 7 883 sq. in.					at 12 000 = 351.9 sq. in.	at 20 000 = 10 610 sq. in.

Floor-Beams.		Stringers.		
Shear.	*Moment.*	*Shear.*	*Moment.*	
D = 47 000 lb.	D = 7 730 000 ft-lb.	D = 8 100 lb.	D = 788 000 ft-lb.	*Web.* 3 Webs. 55 × ⅝ = 103.1
L = 374 000 "	L = 67 300 000 " "	L = 134 000 "	L = 11 470 000 " "	6 Fills. 39 × ⅞ = 204.8
I = 261 000 "	I = 47 000 000 " "	I = 120 900 "	I = 10 350 000 " "	6 Pls. 53 × ⅝ = 198.8 506.7 sq. in.
682 000 lb.	122 030 000 ft-lb.	263 000 lb.	22 608 000 ft-lb.	*Top Flange.* { 6 Ls. 8 × 8 × ⅞ = 79.4g. = 61.7n.
at 12 000 = 56.8 sq. in.	at 82.5 × 20 000 = 74.0 sq. in.	at 12 000 = 21.8 sq. in.	at 65.8 × 20 000 = 17.2 sq. in.	12 Cov. Pls. 18 × 11/16 = 148.5g. = 111.4n.
Web. = 82 × ¾ = 6.15 sq. in.		*W.* = 69 × ½ = 34.5 sq. in.	⅛ web. = 4.3 sq. in.	173.1n.
Flange.	⅛ web. = 7.7	2 Ls. 6 × 6 × ⅝ = 14.2g. = 12.9 sq. in. n.		*Bottom Flange.* { 6 Ls. 8 × 8 × ⅞ = 79.4g.
2 Ls. 6 × 6 × ¾ = 16.9g. = 13.9n.		Full T 38 ft. B, 34 ft., 29 ft., 25 ft., 19 ft.		9 Cov. Pls. 16½ × ¾ = 111.4g. 190.8g.
5 Cov. Pls. 16 × ¾ = 60.0g. = 52.5n.	74.1n.			

TABLE 2.—STRESS SHEET FOR 518-FOOT SPAN.

Member.	D_1.	L_1.	I_1.	$D_1+L_1+I_1$.	L_2.	I_2.	$D_1+L_2+I_2$.	W_1.	Sections.		
a B'	− 2 210	− 3 083	− 693	− 5 986	− 2 254	− 518	− 4 982	17 000 000 ± 477	1 Cov. Pl. 48 × ¾ 4 Ls. 6 × 4 × 11/16	2 Webs. 48 × ½ 2 Webs. 48 × 9/16	
B' C	− 2 070	− 2 872	− 645	− 5 587	− 2 063	− 476	− 4 609	17 000 000 ± 377	4 Ls. 6 × 6 × 11/16 4 Webs. 48 × 11/16	4 Pls. 35 × 11/16 Total 423 sq. in.	
C D E	− 2 000	− 2 793	− 627	− 5 420	− 1 963	− 462	− 4 425	± 170 C D ± 104 D E	1 Cov. Pl. 48 × ¾ 4 Ls. 6 × 4 × 11/16 4 Ls. 6 × 6 × 11/16	4 Webs. 48 × 11/16 4 Pls. 35 × 11/16 Total 321 sq. in.	
E F G	− 2 240	− 3 118	− 702	− 6 060	− 2 156	− 509	− 4 905	± 35 E F ∓ 4 F G	1 Cov. Pl. 48 × ¾ 4 Ls. 6 × 4 × 11/16 4 Ls. 6 × 6 × 11/16	4 Webs. 48 × 11/16 2 Webs. 48 × ⅜ 4 Pls. 35 × 11/16	357 sq. in.
G H I	− 2 380	− 3 322	− 746	− 6 448	− 2 264	− 535	− 5 179	∓ 33 G H ∓ 46 H I	1 Cov. Pl. 42 × ¾ 4 Ls. 6 × 4 × 11/16 4 Ls. 6 × 6 × 11/16	4 Webs. 48 × 11/16 2 Webs. 48 × ½ 4 Pls. 35 × 11/16	369 sq. in.
a b c	+ 1 550	+ 2 165	+ 487	+ 4 202	+ 1 582	+ 364	+ 3 496	± 286 a b ± 449 b c	4 Ls. 6 × 6 × 13/16	= 36.4 g. = 29.9 n.	
c d e	+ 1 550	+ 2 193	+ 492	+ 4 235	+ 1 589	+ 370	+ 3 509	± 597 c d ± 713 d e	4 Webs. 48 × ¾ 2 Webs. 48 × 13/16	= 144.0 g. = 126.0 n. = 78.0 g. = 68.2 n.	224.1 n.
e f g	+ 2 000	+ 2 808	+ 632	+ 5 440	+ 1 957	+ 463	+ 4 420	± 810 e f ± 882 f g	4 Ls. 6 × 6 × 13/16 4 Webs. 48 × ¾ 2 Webs. 48 × 13/16 2 Pl. 35 × 13/16	= 36.4 g. = 29.9 n. = 144.0 g. = 126.0 n. = 78.0 g. = 68.2 n. = 56.9 g. = 50.4 n.	274.5 n.
g h i	+ 2 304	+ 3 236	+ 726	+ 6 266	+ 2 220	+ 521	+ 5 048	± 907 g h ± 932 h i	10 bars. 16 × 2	= 320.0	
C D'	+ 673	+ 1 128	+ 293	+ 2 094	+ 882	+ 219	+ 1 714	....	4 Ls. 6 × 6 × ⅝ 2 Webs. 30 × ½ 4 Webs. 30 × 9/16	= 28.4 g. = 23.4 n. = 30.0 g. = 26.0 n. = 67.5 g. = 58.5 n. 107.9 n.	

TABLE 2.—(*Continued.*)

Member.	D_1.	L_1.	I_1.	$D_1 + L_1 + I_1$.	L_2.	I_2.	$D_1 + L_2 + I_2$.	W_1.	Sections.		
D′ e	+ 525	+ 1 070 − 372	+ 290 − 211	+ 1 885 − 233	+ 800 − 280	+ 217 − 158	+ 1 542 − 88	....	4 Ls. 6 × 6 × ⅝ 4 Webs. 30 × ½ 2 Webs. 30 × 7/16	= 28.4 g. = 23.4 n. = 60.0 g. = 52.0 n. = 28.3 g. = 22.8 n. 98.2 n.	
E F′	+ 512	+ 1 043 − 334	+ 305 − 164	+ 1 860 − 157	+ 764 − 256	+ 230 − 129	+ 1 506 − 43	....	4 Ls. 6 × 6 × ⅝ 2 Webs. 30 × ½	= 28.4 g. = 23.4 n. = 30.0 g. = 26.0 n.	
F′ G	+ 380	+ 985 − 467	+ 301 − 214	+ 1 666 − 427	+ 725 − 362	+ 223 − 167	+ 1 328 − 276	....	4 Webs. 30 × 7/16	= 52.5 g. = 45.5 n. 94.9 n.	
G H′	+ 257	+ 853 − 494	+ 287 − 199	+ 1 397 − 522	+ 623 − 375	+ 216 − 153	+ 1 096 − 359	....	4 Ls. 6 × 6 × ⅝ 6 Webs. 30 × 7/16	= 28.4 g. = 23.4 n. = 78.8 g. = 68.3 n.	
H′ i	+ 128	+ 804 − 621	+ 283 − 238	+ 1 215 − 773	+ 609 − 475	+ 216 − 183	+ 953 − 572			91.7 n	
C c	+ 336	+ 687	+ 369	+ 1 394	+ 494	+ 265	+ 1 095	....	4 Ls. 4 × 4 × ⅝ 4 Webs. 30 × ¾	= 18.4 g. = 15.9 n. = 90.0 g. = 78.0 n. 93.9 n.	
E e	− 142	− 553 + 494	− 167 + 239	− 862 + 638	− 421 + 370	− 128 + 183	− 691 + 458	....	4 Ls. 6 × 6 × ¾	= 33.8	
G g	− 70	− 553 + 563	− 183 + 222	− 786 + 739	− 452 + 419	− 159 + 171	− 681 + 543	....	2 Webs. 30 × 13/16	= 48.8 82.6	
I i	− 82	....	....	− 82	....	....	....	....	4 Ls. 4 × 4 × ⅜ 2 Webs. 30 × 7/16	= 11.4 = 26.3 37.7	
Sub-Hanger	+ 127	+ 374	+ 261	+ 762	+ 264	+ 164	+ 555	....	4 Ls. 6 × 4 × 9/16 2 Webs. 30 × ⅝	= 21.2 g. = 19.0 n. = 37.5 g. = 32.5 n.	51.5 n.
B′ C *C D′*	− 148	− 263	− 184	− 595	− 185	− 115	− 448	....	4 Ls. 4 × 4 × ⅝ 2 Webs. 22 × ⅝	= 18.4 = 27.5 45.9	
F′ e	− 132	− 236	− 165	− 533	− 166	− 103	− 401	....	do.		
H′ g	− 128	− 230	− 160	− 518	− 162	− 101	− 391	....	do.		
D D′ *F F′-H H′*	− 82	....	....	....	....	....	....	....	4 Ls. 4 × 4 × ⅜ 2 Webs. 30 × 7/16	= 11.4 = 26.3 37.7	

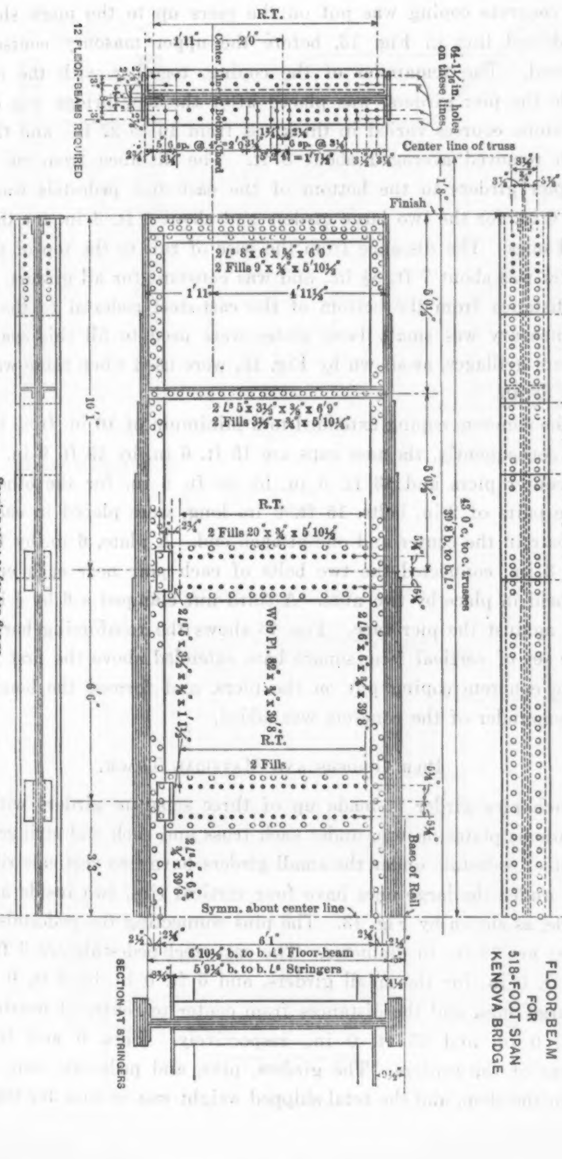

FLOOR-BEAM
FOR
518-FOOT SPAN
KENOVA BRIDGE
12 FLOOR-BEAMS REQUIRED
SECTION AT STRINGERS
Center line of truss
Finish
Base of Rail
Symm. about center line
Spacing same as for top flange
R.T.
2 Fills 20" x ⅜" x 5' 10½"
6' 10½" b. to b. L^s Floor-beam
5' 9½" b. to b. L^s Stringers
43' 0" c. to c. trusses

FIG. 11.

this concrete coping was put on the piers up to the mark shown by the dotted line in Fig. 13, before the upper masonry courses were removed. The remainder of the coping, together with the concrete inside the pier girders, was placed after the new bridge was erected. The stone courses varied in thickness from 16 to 22 in., and the total depth removed averaged about 8 ft. The distance from the top of the pier girders to the bottom of the cast-steel pedestals was about 6 ft. 0 in. for the two large girders and about 5 ft. 3 in. for the eight small ones. The distance from the base of rail to the top of the pier girders was about 7 ft. 8½ in., and was constant for all girders. When the distance from the bottom of the cast-steel pedestal to the top of the masonry was small, loose plates were used to fill this space, and I-beam grillages, as shown by Fig. 13, were used when there was sufficient depth.

The concrete coping extended to a minimum of 16 in. from the pier face, consequently, the new caps are 15 ft. 6 in. by 49 ft. 9 in. for the two center piers and 13 ft. 6 in. by 46 ft. 6 in. for the other four. Three pairs of 2-in. bolts, 15 ft. 0 in. long, were placed in each pier, one pair in the center and one at each end. A plate, 6 in. by 1 in. by 4 ft. 3 in., connected the two bolts of each pair near each end, and was held in place by two nuts. A third nut clamped a 6 by ½ by 6-in. plate against the pier face. Fig. 13 shows the reinforcing bars. The lower set of vertical ¾-in. square bars extended above the first portion of the concrete coping put on the piers, and formed the bond when the remainder of the concrete was added.

Pier Girders and Material Track.

Each pier girder is made up of three separate girders with 1-in. continuous plates on top, under each truss and each end stringer. The cast-steel pedestals under the small girders have two verticals ribs, and those under the large ones have four vertical ribs, two inside and two outside, as shown by Fig. 13. The pins connecting the pedestals to the girders are 18 in. in diameter. The cast-steel pedestals are 3 ft. 8 in. by 8 ft. 0 in. for the small girders, and 6 ft. 0 in. by 9 ft. 0 in. for the large ones, and the distances from center to center of bearings are 33 ft. 0 in. and 35 ft. 0 in., respectively. Figs. 9 and 10 show sections of the girders. The girders, pins, and pedestals were assembled in the shop, and the total shipped weight was 88 tons for the small

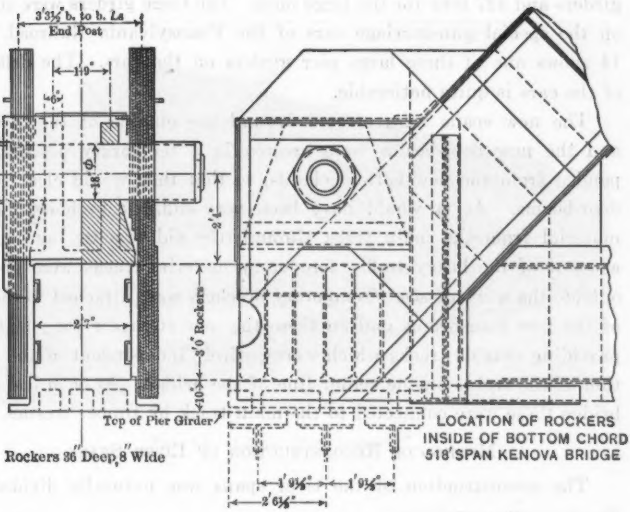

3' 3½" b. to b. Ls
End Post
1' 9"
6"
1' 8"
2' 4"
2' 7"
3' 0" Rockers
10"
Top of Pier Girder
Rockers 36" Deep, 8" Wide
1' 9⅛"
1' 9⅛"
2' 6⅛"
LOCATION OF ROCKERS
INSIDE OF BOTTOM CHORD
518' SPAN KENOVA BRIDGE

FIG. 12.

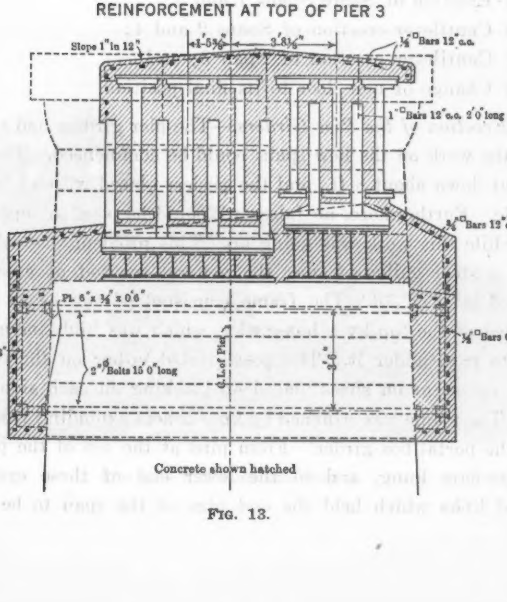

REINFORCEMENT AT TOP OF PIER 3
Slope 1" in 12"
1-5⅝"
3-8½"
½" □ Bars 12" c.c.
1" □ Bars 12" c.c. 2' 0" long
¾" □ Bars 12" c.c.
Pl. 6" x ½" x 0' 6"
½" □ Bars 6" c.c.
Pl. 6" x 1" x 4' 3"
2" ⌀ Bolts 15' 0" long
C. L. of Pier
Concrete shown hatched

FIG. 13.

girders and 127 tons for the large ones. The large girders were shipped on the special gun-carriage cars of the Pennsylvania Railroad. Fig. 14 shows one of these large pier girders on the cars. The deflection of the cars is quite noticeable.

The new spans were erected around the outside of the old ones, and the new floor-beams were erected in a temporary position, suspended from the new bottom chords, so that they would clear the old floor-beams. As it would have been very difficult to hoist the new material from the main track through the old bracing, and also on account of the heavy traffic, temporary material tracks were provided outside the new trusses. Temporary brackets were attached to the ends of the new floor-beams and on these the new stringers were laid, thus providing outside tracks which were entirely independent of the operation of the trains on the center line of the bridge. At each end of the bridge these were connected to the main track by timber trestles.

Method of Reconstruction of River Spans.

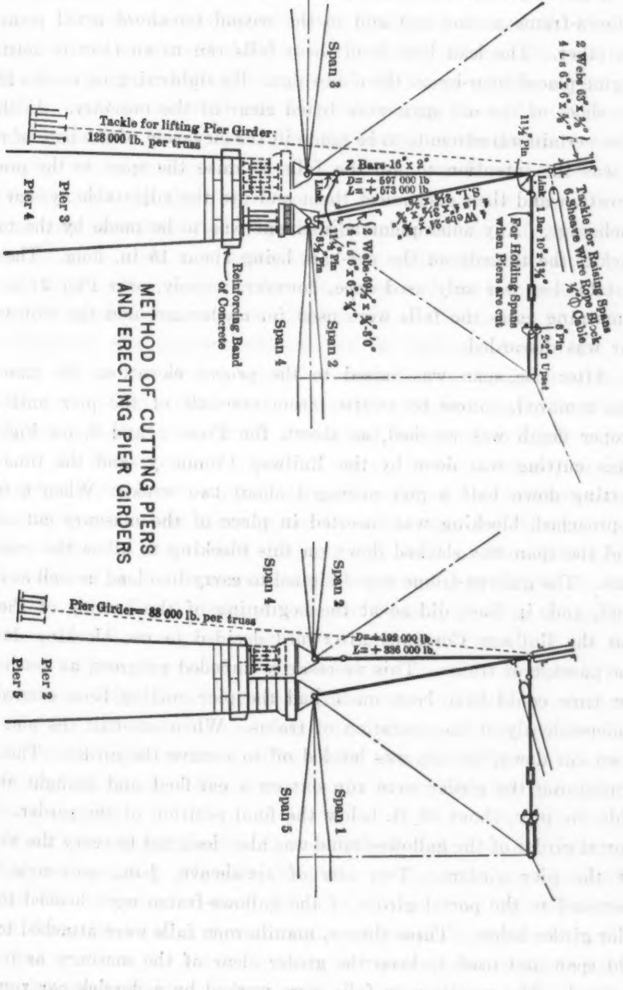
The reconstruction of the river spans was naturally divided into five separate operations:

(1) Erection of the pier girders;
(2) Erection of Shore Spans 1 and 5;
(3) Cantilever erection of Spans 2 and 4;
(4) Cantilever erection of Span 3; and
(5) Change of new floor to its final position.

1.—Erection of the Pier Girders.—The pier girders had to be placed before the work on the new spans could be commenced. The piers had to be cut down about 8 ft., and the girders placed without interruption of traffic. Furthermore, no falsework could be used to support the old spans while this work was going on. This problem was solved by the use of a steel gallows-frame, the general method of erection being indicated by Fig. 16. The frame consisted of two heavy steel posts connected at the top by a box-girder, which was high enough to allow trains to pass under it. The posts rested either on the end pins of the old spans, or on shoes placed on blocking on each side of the old shoes. The frame was stiffened by knee-braces extending from the posts up to the portal box-girder. From pins at the top of the posts, 16 by 2-in. eye-bars hung, and to the lower end of these eye-bars were fastened links which held the end pins of the span to be supported.

FIG. 14.—LARGE PIER GIRDER.

FIG. 15.—END POST FOR SPAN 3.



METHOD OF CUTTING PIERS AND ERECTING PIER GIRDERS

FIG. 16.

From the pin at the top of the post, a 10 by $1\frac{3}{8}$-in. horizontal adjustable eye-bar extended back to an equalizer which, in turn, connected to the pin at the hip point of the anchor span.

A set of six-sheave, wire-rope falls was connected to the top of the gallows-frame at one end and to the second top-chord panel point at the other. The lead line from these falls ran to an electric hoisting engine placed near-by on the old bridge. By tightening up on the falls, the shoes of the old span were lifted clear of the masonry. As there were certain adjustments to be made in the elevation of the base of rail, it was the intention to use the falls to raise the span to the proper elevation and then disconnect them and use the adjustable eye-bar for anchorage. Any subsequent adjustment was to be made by the turnbuckle, the threads on the eye-bars being about 15 in. long. The adjustable bar was only used once, however, namely, over Pier 2; in the remaining cases the falls were used for anchorage, and the adjustable bar was discarded.

After the span was raised to the proper elevation, the masonry was removed, course by course, from one-half of the pier until the proper depth was reached, as shown for Piers 2 and 5, on Fig. 16. This cutting was done by the Railway Company, and the time for cutting down half a pier averaged about two weeks. When a train approached, blocking was inserted in place of the masonry cut away, and the span was slacked down on this blocking to allow the train to pass. The gallows-frame was designed to carry live load as well as dead load, and, in fact, did so at the beginning of the cutting of Pier 1, but the Railway Company afterward decided to use blocking during the passage of trains. This necessarily impeded progress, as much better time could have been made had the pier cutting been carried on independently of the operation of trains. When one-half the pier had been cut down, the top was leveled off to receive the girder. The cars containing the girder were run out on a car-float and brought alongside the pier, about 85 ft. below the final position of the girder. The portal girder of the gallows-frame was also designed to carry the weight of the pier girders. Two sets of six-sheave, $\frac{3}{4}$-in., wire-rope falls fastened to the portal girder of the gallows-frame were hooked to the pier girder below. Three-sheave, manila-rope falls were attached to the old span and used to keep the girder clear of the masonry as it was hoisted. The manila-rope falls were worked by a derrick car run out

on the bridge and the wire-rope falls by the electric engine used to raise and lower the gallows-frame. This engine had four drums and eight spools. The wire-rope falls were 19 ft. 0 in. apart, wide enough to straddle the track and yet sufficiently narrow to fall inside the trusses. Trains were stopped while the girder was hoisted, the blocking being kept under the old shoes until the girder reached the pier level. Then the top of the gallows-frame was pulled back a little, so that the blocking could be removed and the girder swung into place. The top of the girder was about 2 ft. below the bottom of the old shoe, so that blocking was placed on which the old span would rest after being slacked down. As there were no end floor-beams, the end stringers of one span were supported by pockets attached to those of the adjacent span while the piers were being cut. The gallows-frame was then reversed to the other span on the same pier, and the remainder of the pier cutting was done, as shown for Piers 3 and 4, on Fig. 16.

Where it was impracticable to let the gallows-frame posts rest directly on the end pins, a special cross-girder was designed to fasten to the post. This girder, which was wide enough to straddle the old end post, rested on two small shoes, and was used at Piers 1, 3, 4, and 6. At Piers 1 and 6 (the end piers), it was necessary to erect a double timber bent to support the gallows-frame, and as there was no adjacent span at these piers to use for anchorage, the falls at Pier 1 were attached to the viaduct columns and those at Pier 6 to anchor rods fastened to the abutments of a road crossing about 300 ft. from the latter pier. Fig. 17 is an end view of Span 1, showing the gallows-frame with the cross-girders at the bottom of the post and the falls fastened back to the viaduct. It also shows the falls used to lift the pier girder. The pier girders for Span 1 were delivered on shore and skidded to the bottoms of the piers. The remaining pier girders, however, were brought to the foot of their respective piers on floats, as mentioned previously. Fig. 19 shows the pier girder for Pier 5 being lifted into place.

Work was started on Pier 1, and Piers 2, 3, 4, 6, and 5 were completed in succession. No difficulties of any moment were experienced, except that it proved a more difficult job than was expected to take down the gallows-frame from one pier and erect it on another. This was due partly to the old spans being in the way and partly to having to work between trains.

2.—Erection of Shore Spans 1 and 5.—After the pier girders were placed under Span 1, the erection of the new work on this span was started. It was originally intended to erect Spans 1 and 5 simultaneously, then Spans 2 and 4, and finally Span 3, the two erection gangs working independently of each other. As the piers for Span 1 were ready for the new steel long before those for Span 5, it was impossible to carry out this part of the programme. Fig. 21 shows the general method of erection of Spans 1 and 5.

Falsework was erected beneath the span, as shown by Fig. 21, and the new floor-beams were brought out and hung from the old ones by 3-in. rods. As these floor-beams were erected, they were blocked temporarily on falsework so as not to put too much load on the old span while it was carrying traffic. Pony bents were erected on each side of the new floor-beams, and the old stringers were blocked up on these bents. The rivets connecting the old stringers to the old floor-beams were cut out, letting the falsework carry the live load, track, and stringers. The hanger rods holding up the new floor-beams were then screwed up until the latter showed the right amount of camber, which had been determined previously. The conditions governing this will be given later.

I-beams were placed on the top chord in the two center panels and blocked up at the panel points so as not to allow any bending in the top chord from the weight of the traveler. The track was laid on these I-beams and the small traveler was erected, a 6-ton ginny-wink being used. This traveler consisted of a 30 by 34-ft. frame, with one 16-in. steel wheel at each corner and one in the center of each sill; 12 by 12-in. posts were placed at each corner and braced together. A lifting beam, composed of two 24-in., 100-lb. I-beams connected by diaphragms, was put at each end of the traveler on top of the posts. One 50-ft. boom was placed at each end of the traveler, resting on a foot-block in the center of the cross-sill. A four-drum, eight-spool, 100-h.p., electric engine was placed on the floor of the traveler, which floor consisted of seven 15-in., 42-lb. I-beams resting on the traveler sills. After the traveler was erected, it put in the remainder of the falsework on top of the span.

The new stringers were erected on the brackets on the ends of the floor-beams, and temporary tracks were laid on these stringers throughout the entire length of the span, thus providing means for bringing the

FIG. 17.—GALLOWS FRAME AT PIER 1.

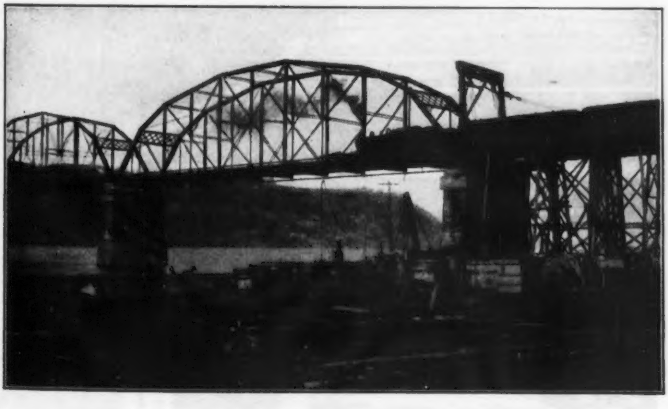


FIG. 18.—GALLOWS FRAME HOLDING SPAN 1.

FIG. 19.—PIER GIRDER BEING LIFTED FROM BARGE.

FIG. 20.—PANEL POINT *j* BEFORE CLOSING.

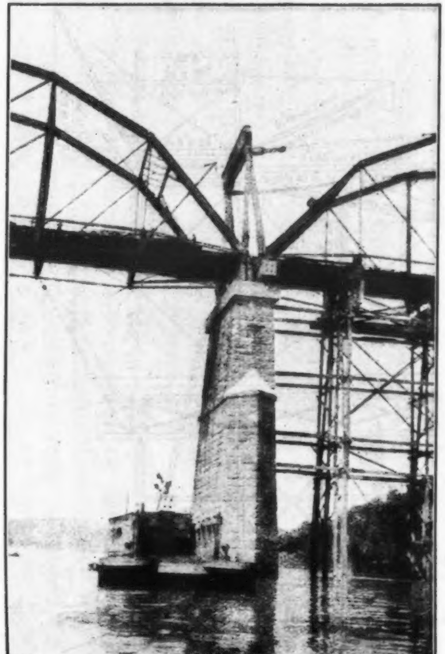


FIG. 19.—PIER GIRDER BEING LIFTED FROM BARGE.

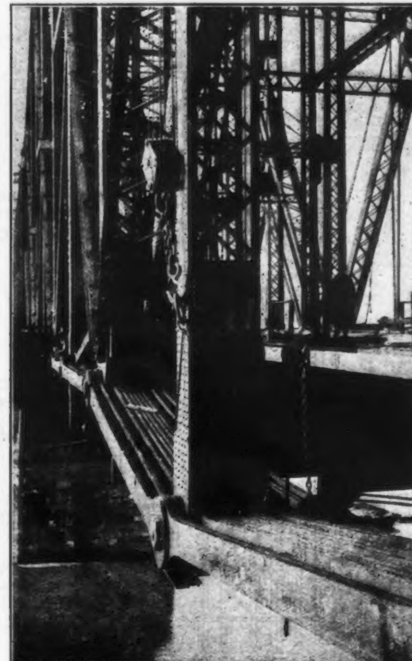


FIG. 20.—PANEL POINT j BEFORE CLOSING.

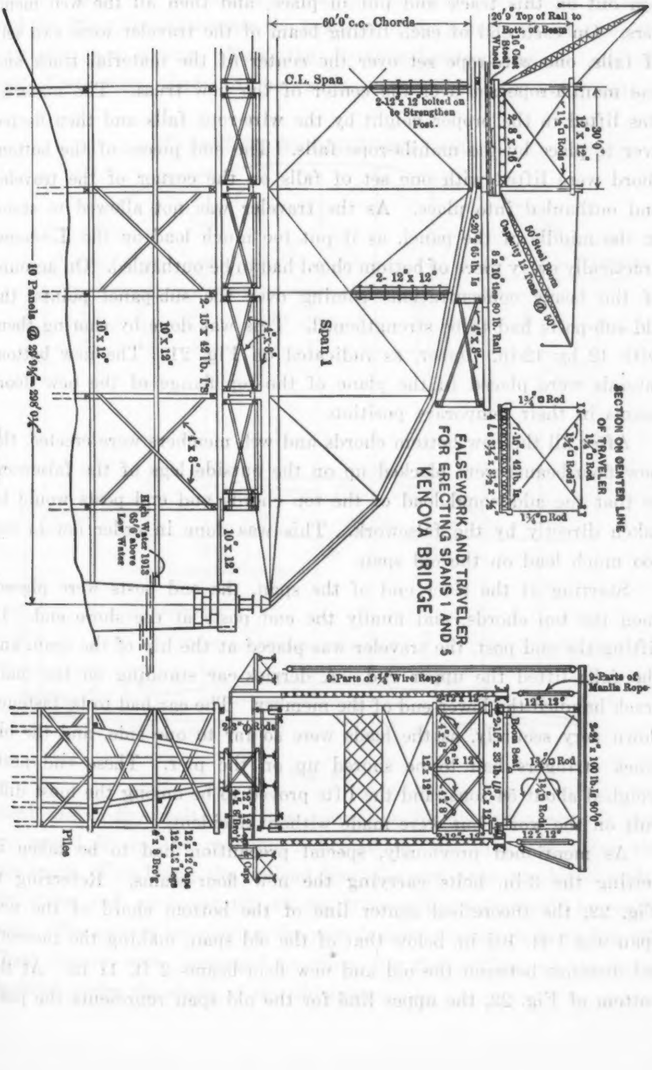

FALSEWORK AND TRAVELER FOR ERECTING SPANS 1 AND 5 KENOVA BRIDGE
SECTION ON CENTER LINE OF TRAVELER
60'0" c.c. Chords
26'9" Top of Rail to Bott. of Beam
C.L. Span
2-12" x 12" bolted on to Strengthen Post.
2- 12" x 12"
Span 1
60' Steel Boom
10 Panels @ 28'9¾" = 288'0¼"
High Water 1913
9-Parts of ⅝" Wire Rope
9-Parts of Manila Rope
12" x 12"
Piles

FIG. 21.

new material out to its proper place in the bridge entirely independent of the main track. Run-offs from these material tracks connected with the viaduct about 100 ft. from Pier 1. All the bottom chords were run out on this track and put in place, and then all the web members. On each end of each lifting beam of the traveler were two sets of falls, one wire-rope set over the center of the material track and one manila-rope set over the center of the new truss. The material was lifted to the proper height by the wire-rope falls and then fleeted over to place by the manila-rope falls. The end pieces of the bottom chord were lifted with one set of falls on the corner of the traveler and outhauled into place. As the traveler was not allowed to stand in the middle of the panel, as it put too much load on the I-beams, practically every piece of bottom chord had to be outhauled. On account of the heavy concentrations coming over the sub-panel points, the old sub-posts had to be strengthened. This was done by shoring them with 12 by 12-in. timber, as indicated on Fig. 21. The new bottom laterals were placed in the plane of the top flange of the new floor-beams in their temporary position.

After all the new bottom chords and web members were erected, the new floor-beams were blocked up on the outside legs of the falsework so that the additional load of the top chords and end posts would be taken directly by the falsework. This was done in order not to put too much load on the old span.

Starting at the river end of the span, the end posts were placed, then the top chords, and finally the end posts at the shore end. In lifting the end post, the traveler was placed at the hip of the span, and the falls lifted the upper end. A derrick-car standing on the main track handled the lower end of the member. The car had to be fastened down very securely, as the loads were so far to one side, and the old track stringers had to be shored up on the pier. These end posts weighed about 54 tons, and the lifts proved to be among the most difficult on the work, but were made without accident.

As mentioned previously, special precaution had to be taken in setting the 3-in. bolts carrying the new floor-beams. Referring to Fig. 22, the theoretical center line of the bottom chord of the new span was 1 ft. 10$\frac{3}{8}$ in. below that of the old span, making the theoretical distance between the old and new floor-beams 2 ft. 11 in. At the bottom of Fig. 22, the upper line for the old span represents the posi-

tion of the center line of the bottom chord after the stringers had been disconnected. The other line represents the position of the bottom chord when the old span carried the new one. The upper line for the new span was the center line of the bottom chord under no load, and the other line represents the position of the bottom chord after the new span was swung. Now, the distance between the old and the new floor-beams had to be such that when the old bottom

DEFLECTION OF OLD SPANS 1 AND 5

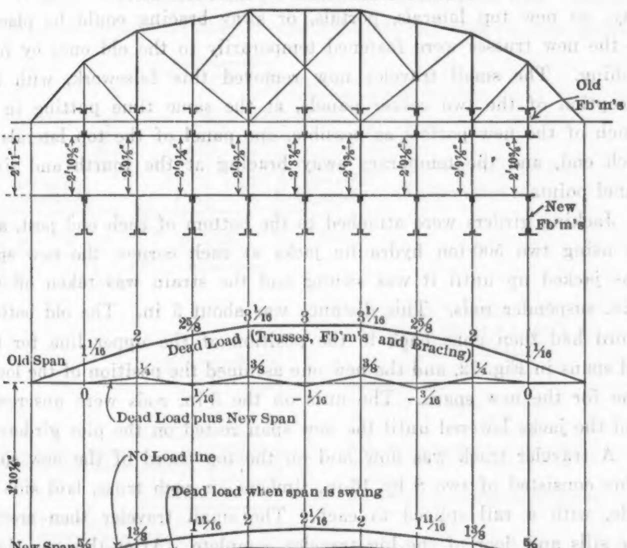


FIG. 22.

chord was in the position of the lower line for the old span, the new one would be in the position indicated by the upper line for the new span. This being the no-load line, the new span would be under zero stress and, consequently, all the truss connections could be easily made. The figures for placing the new floor-beams are given in Fig. 22. These figures were increased to allow for the settling of the wood blocking under the shoes of the old span.

The total actual deflection in the center of the span at the time the new floor-beams were blocked up on the outside legs of the falsework was about 2½ in. below the dead-load camber, or ½ in. below a horizontal line drawn from center to center of the end pins. Wedges had to be provided between the pony bents and old stringers so that the latter could be raised and lowered with the movements of the span, otherwise the old floor-beams would bear on the tops of the stringers and put more load on the falsework.

On account of the falsework on top of the old span being in the way, no new top laterals, portals, or sway bracing could be placed, so the new trusses were fastened temporarily to the old ones by rope lashing. The small traveler now removed this falsework, with the exception of the two center panels, at the same time putting in as much of the new portals as possible, one panel of the top laterals at each end, and the temporary sway bracing at the fourth and sixth panel points.

Jacking girders were attached to the bottom of each end post, and by using two 500-ton hydraulic jacks at each corner, the new span was jacked up until it was swung and the strain was taken off the 3-in. suspender rods. This distance was about 5 in. The old bottom chord had then gone back to the position of the upper line for the old spans in Fig. 22, and the new one assumed the position of the lower line for the new spans. The nuts on the 3-in. rods were unscrewed and the jacks lowered until the new span rested on the pier girders.

A traveler track was now laid on the top chord of the new span. This consisted of two 8 by 16-in. timbers on each truss, laid side by side, with a rail spliced to each. The small traveler then erected the sills and floor of the big traveler complete. After this was done, the small traveler was jacked up and extensions were put under it so that it could run on the track laid on the new top chord. The remainder of the falsework was removed from the top of the old span, and the remaining new top laterals and struts were placed. The old stringers were then reconnected to the old floor-beams by bolts, thus letting the old span again carry the live load. The small traveler placed a 12-ton, steel derrick with a 65-ft. boom on the deck of the large traveler. This derrick raised the remainder of the large traveler and was then taken down by the small traveler. The electric engine, which up to this time had been on the small traveler, was transferred

to the large traveler, and a two-drum, four-spool, steam engine was placed on the small traveler to do what little hoisting remained for this traveler. The large steel traveler was then rigged up complete. Fig. 23 shows sections of this traveler. The masts, booms, stiff legs, and cast-steel foot-blocks were standard parts of a 65-ton derrick, the remainder had been made especially for this bridge. The traveler could have been made somewhat lighter if it had been designed throughout for this bridge, but as the 65-ton derricks would be needed later for other work, it was decided to use them at Kenova as well. Aside from the great weight of the traveler (423 000 lb.) and its very great

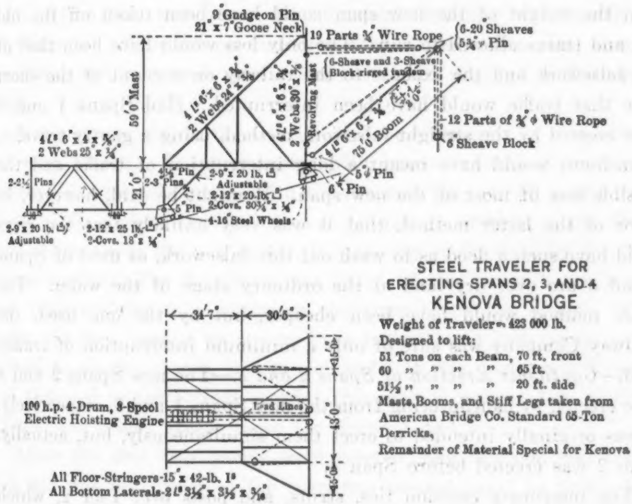


Fig. 23.

lifting capacity, the only other unusual feature was the arrangement for keeping the revolving mast plumb when working on the inclined top chords of Span 3. This was taken care of by a long leg in the back and telescoping braces. In order to raise the rear end of the traveler, the booms were hooked to the old span ahead. At the proper height, pins were driven in the back legs and telescoping braces. To work satisfactorily, a traveler or derrick with revolving masts should have these masts plumb.

The work of removing the falsework under the span was commenced, and the 15-in. I-beams, on which the pony bents had rested,

were placed on top of the new floor-beams. The old span was blocked up on these I-beams, and the old trusses and bracing were removed, using both travelers. The old floor system was not removed at this time, but was allowed to remain blocked up on the I-beams until all the new spans were erected. Span 5 was erected in the same way, but it was not started until some time after Span 1 had been put in place.

If there had been a flood in the river after the erection of the new trusses had been commenced, and this flood should have taken out the falsework, traffic would have been interrupted only for the length of time necessary to complete the erection of the new span, for then the weight of the new span could have been taken off the old one and trains allowed to run. The only loss would have been that of the falsework and the expense to the railway on account of the short time that traffic would have been interrupted. Had Spans 1 and 5 been erected by the straight-falsework method, using a gantry traveler, a wash-out would have meant a long interruption of traffic and the possible loss of most of the new span. It might be said, however, in favor of the latter method, that it was very unlikely that the river could have such a flood as to wash out this falsework, as most of Spans 1 and 5 are over dry land at the ordinary stage of the water. The latter method would have been cheaper, but by the one used, the Railway Company was sure of only a minimum interruption of traffic.

3.—Cantilever Erection of Spans 2 and 4.—The new Spans 2 and 4 were erected by cantilevering from the new Spans 1 and 5, respectively. It was originally intended to erect them simultaneously, but, actually, Span 2 was erected before Span 4.

The temporary erection ties, struts, and posts over Pier 2, which would connect the new Spans 1 and 2, shown by dotted lines in Fig. 7, were erected by the large traveler. The distance between the two hip points of adjacent spans was about 125 ft., and the ties were supported at each panel point by vertical posts. The total section in each tie was 225 sq. in., and, as the large traveler had to cross them, the ties were made deep enough to be good for this load in bending. The direct tension in each tie was about 4 000 000 lb. Span 1 acted as the anchor arm, and, for additional counterweight, 286 000 lb. of rails were placed on its shore end and the small traveler was run as far back toward the shore on this span as possible to serve also as a counterweight.

After the erection ties were in position, the end posts for Span 2, each weighing about 54 tons, were brought out on the material track and hoisted into place, one boom being used for each post. The shoes of these posts were put in in the material yard. The hitch for lifting them was placed so that they hung in the correct position after being hoisted clear of the push-cars. Temporary adjustable eye-bars, which fastened to the center of the end post and to the top of the erection post over Pier 2, were now connected up and held the end post until the remainder of the erection ties, the first piece of the bottom chord, and the first two panels of the web members, were put in by the traveler. After this was done, the adjustable eye-bars were removed. At the point where the erection ties were spliced, there were slotted pin-holes, and into each was inserted a round pin and a **D**-shaped plug.

A track was laid on the erection ties, and the traveler was moved out over Pier 2. The floor-beams with brackets attached were run out on the material track. With the brackets the over-all length was about 67 ft., and at first it seemed to be quite a problem to get these floor-beams under the new span. It was very easily done by the following method: The floor-beam was lifted off the car with one boom and slacked down to a barge on the river below the bridge. The barge was then pushed up stream by a tugboat, and both booms were hooked to the floor-beam, one at each end. It was then easily hoisted and fastened to the bottom chord. This method was used in placing all the floor-beams except those for Spans 1 and 5.

After the first two panels were erected, and the material track was laid, the remainder of the span was erected panel by panel, the large traveler creeping out on the top chord. As much as possible of the new bracing was placed at this time. The erection tie was shortened $7\frac{1}{4}$ in. in order to be sure that the river end of Span 2 would clear the pier girder; this clearance was about 3 in. A typical cross-section of Span 2, during and after erection, is shown at the right of Fig. 24.

Jacks were then placed under jacking girders attached to the shore end of Span 1 and the river end of Span 2, and both travelers were run over Pier 2 so as to put as little load as possible on the jacks. The jacks were pumped and the ends of the spans raised until the stresses in the trusses were reversed and there was zero stress in the

erection ties. At this time each span was acting as a simple span, and the D-shaped plugs previously mentioned were knocked out of the slotted holes. Each slot was so long that, after the spans were lowered, the erection ties could still be used for the travelers to cross from one span to the other. The total jacking was about 25 in., Span 1 being jacked 16 in. and Span 2, 9 in. This latter span could not be jacked any more on account of fouling old Span 2. During this jacking, the track was broken and the traffic interrupted, but the whole operation, from the time the track was broken until it was reconnected, did not take more than 2 hours. The spans were then lowered on the pier girders.

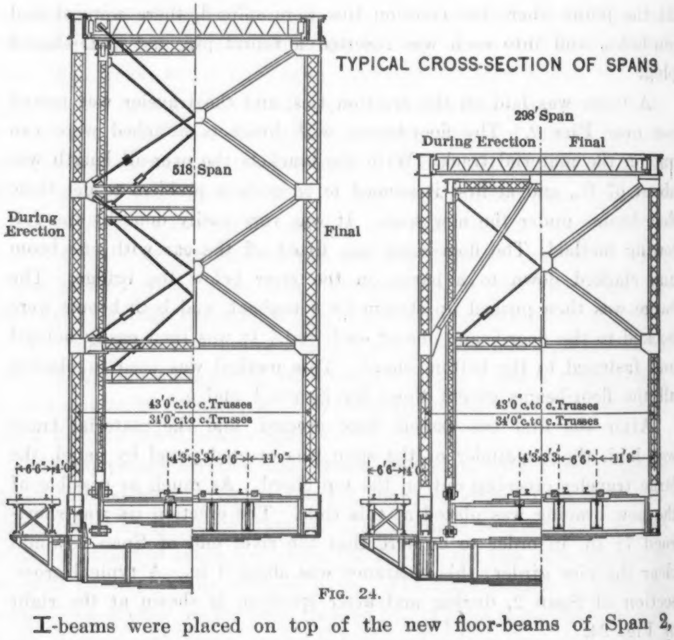


FIG. 24.

I-beams were placed on top of the new floor-beams of Span 2, and on these old Span 2 was blocked up, and then the trusses and bracing were taken down. These trusses were allowed to remain on new Span 2 so as to be used as a counterweight when the span acted as the anchor span during the erection of new Span 3. The method of erecting Span 4 was similar to that used for Span 2.

4.—Cantilever Erection of Span 3.—The small traveler crossed the erection ties to Span 2, removing the ties as it crossed. These ties were designed so that many of the pieces could be used again between Spans 2 and 3, and such as could be used were erected in position over Pier 3 by the large traveler. (See dotted lines in Fig. 7.) The small traveler was placed as near as possible to the shore end of Span 2 to serve as a counterweight, and an additional counterweight was provided by the rails which had previously been used on Span 1.

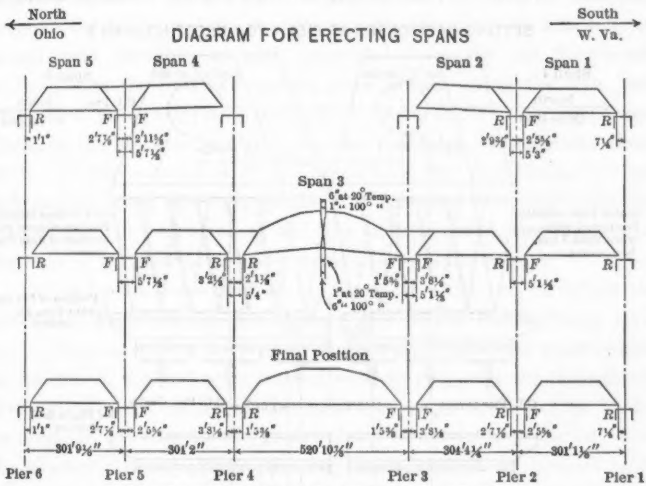


FIG. 25.

Span 3 was erected by cantilevering each half from new Spans 2 and 4, and then joining the two halves in the center. This is shown on Fig. 25. The south end of Span 3 was fixed and the north end on rockers. The adjacent or south end of Span 4 was also on rockers, so that there were two rocker ends over Pier 4. The purpose of this will appear later. The joint in Span 3 was made at the center of the top chord, but at the sub-panel point next to the center in the bottom chord (see Fig. 7) the four center panels of the bottom chord were made of eye-bars. As the bottom chords shortened under erection compression, and as there would be still more contraction in cold weather, new Span 4 had been erected 6 in. south of its final position.

That was done so as to be sure that at a temperature of 20° there would be a minimum lap of 1 in. Furthermore, to insure a minimum gap of 1 in. in the top chord at 100°, the erection ties were shortened 9½ in. over Piers 3 and 4. In order to close Span 3, the shore ends of Spans 2 and 4 were to be jacked until the top chords of Span 3 touched, closing the gap. Then, as jacking continued, the bottom chord would straighten out by the longitudinal movement of Span 4 and the north half of Span 3. On further jacking, the stresses would reverse, and finally all three spans would be acting as

SETTING OF ROCKERS AT PIER 4 FOR ERECTING SPAN 3

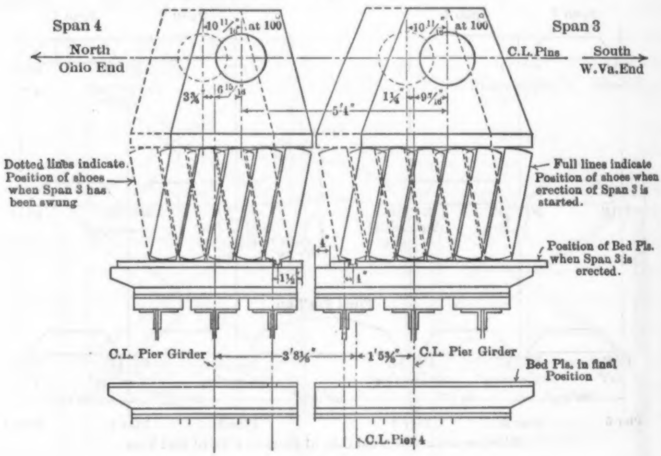


FIG. 26.

simple spans and Span 4 would have moved back to its final position. The rockers on Pier 4 were adjusted by sliding plates so that the inclination south before jacking would about equal the inclination north after Span 3 was closed. Fig. 26 shows the setting of the rockers over Pier 4.

After the erection ties were placed, the end post for Span 3 was brought out. This end post was about 105 ft. long and weighed 95 tons by itself, or about 105 tons when the shoe and pin were attached. It is shown in Fig. 15. As the capacity of each boom was only 65 tons, these posts had to be handled with both booms of the traveler. This was done by a balance beam hooked to the post at such a point

that, when the end post was lifted clear of the cars, it hung at the proper inclination. The balance beam served to distribute the load equally between the two booms. In order to place the end posts, the traveler had to stand directly over Pier 3, and, as the material track extended only as far as the pier, the booms could not reach the balance beam unless some provision was made for extending the material track farther out. Also, it was practically impossible to run the end post out on the main track and hoist it between the portals of old Span 3 and new Span 2. To get the end post in such a position that both booms could be hooked to it, two new floor-beams with brackets attached were brought out and suspended from the old floor-beams of Span 3 at the first and second panel points, using the 3-in. rods which were previously used similarly in Span 1. New stringers were placed on the brackets, and thus the material track was extended about 65 ft. As the connection of the brackets to the floor-beams was not sufficient to stand the load of the end post and the cars when the bracket was not connected to the new bottom chord, the brackets at the ends of the floor-beam were connected by 3-in. rods.

The end posts were shipped on three cars, the two on which the load rested being special gun-carriage cars of the Pennsylvania Railroad. These cars have no floor in the center, so that the gusset-plates at the ends of the end posts were allowed to project down through the open space in the center of the car. The posts could not be loaded on ordinary flat cars as the gusset-plates would have projected above the clearance lines. When the end posts arrived at Kenova, they were not unloaded, but one end was lifted high enough for the idler car to be taken out. The end car was then put back, but in such a position that the center of gravity of the end posts was as near as possible to the forward end of the car. The posts were brought out on the material track and placed by the traveler without any trouble. Temporary adjustable eye-bars, similar to those used during the erection of the end post for Span 2, were used.

After the first two panels of Span 3 were erected the other panels up to the center were erected by methods similar to those used for Span 2. The left side of Fig. 24 shows a typical cross-section of Span 3, during and after erection. In this case, however, the traveler had to climb the curved top chord. To pull the traveler up the inclined top chord of the new span, two sets of six-sheave, wire-rope falls were

used, fastened to the top of old Span 3. Also, two sets of four-sheave, manila-rope falls were put on, to catch the traveler in case anything gave away. After the traveler was pulled up to its proper place, the brackets were fastened to the top chord behind the front shoes. The falls were then slacked off, and the brackets prevented the traveler from running down the incline.

Attention has been called to the fact that the erection ties were shortened 7¼ and 9¼ in. over Piers 2 and 3, respectively, for certain purposes. There was another feature to be considered, which necessitated this shortening being kept to a minimum. The actual eleva-

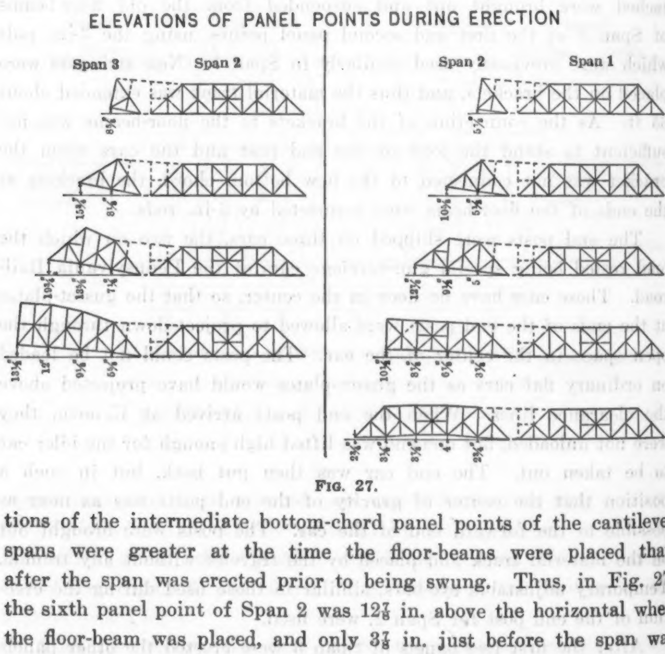


FIG. 27.

tions of the intermediate bottom-chord panel points of the cantilever spans were greater at the time the floor-beams were placed than after the span was erected prior to being swung. Thus, in Fig. 27, the sixth panel point of Span 2 was 12⅞ in. above the horizontal when the floor-beam was placed, and only 3⅜ in. just before the span was swung. Also, the sixth panel point of Span 3 was 19¾ in. above the horizontal when the floor-beam was placed, and only 12 in. just before the span was closed. The old Spans 2, 3, and 4, were carrying traffic during this period, therefore it was necessary to drop the new floor-beams sufficiently to clear the old spans. At the same time, it was

not advisable to go down too far, as the bottom lateral system in temporary position was in the plane of the top of the new floor-beams, and hence below the new bottom chords.

To determine as nearly as possible the actual deflections of the old spans under traffic, a set of levels was taken on the bottom-chord panel points of all five old spans, first under dead load and

Camber of Old Spans.

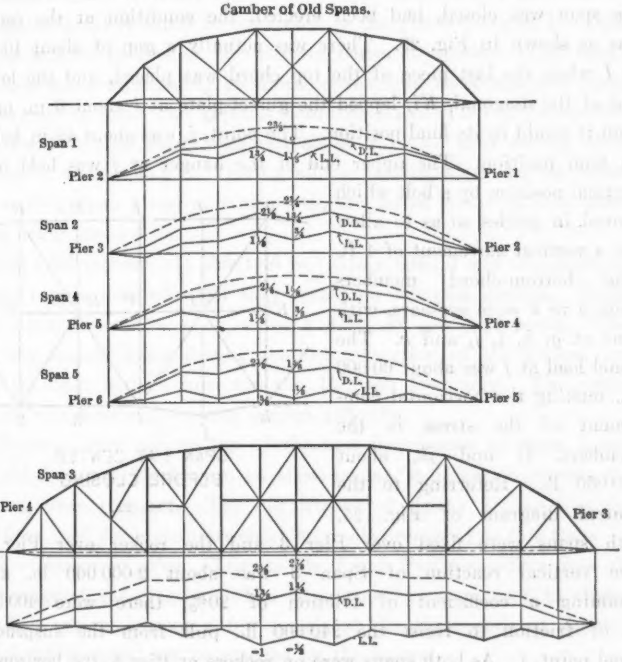
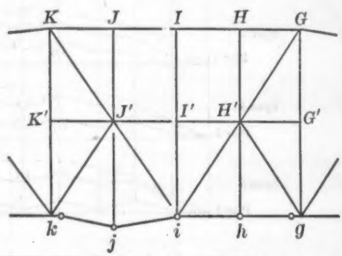


FIG. 28.

second under an actual live load, a train being stopped on the bridge for the purpose. The results of these tests are plotted in Fig. 28. The dotted lines represent the theoretical dead-load line, the upper full lines are the actual dead-load lines, and the lower full lines are the positions of the bottom chords under dead and live loads. It will be seen from Fig. 28 that although the bottom chord of the 298-ft. spans did not go below the horizontal under live load, the

518-ft. span near the center went down as much as 1 in. As a matter of precaution, therefore, the floor-beams near the center of the 518-ft. span were dropped from 4 to 8 in. more than had been intended originally. It was not necessary to place the three center floor-beams of Span 3 until after the new span was swung, as there was no material track in the four center panels of this span.

When all the trusses and bracing which could be placed before the span was closed, had been erected, the condition at the center was as shown in Fig. 29. There was actually a gap of about 10 in. at *I* when the last piece of the top chord was placed, and the lower end of the diagonal, *Ki*, lapped the gusset-plate at *i* about 6 in. more than it would in its final position. The point, *j*, was about 45 in. below its final position. The upper end of the hanger at *j* was held in a vertical position by a bolt which moved in guides so as to allow for a vertical movement of 4 ft. The bottom-chord members from *g* to *k* were eye-bars, with pins at *g*, *h*, *i*, *j*, and *k*. The panel load at *j* was about 60 000 lb., making the horizontal component of the stress in the members, *ij* and *jk*, about 240 000 lb. Referring to the central diagram of Fig. 25, both spans were fixed over Pier 3 and the rocker over Pier 4. The vertical reaction of Span 3 was about 2 000 000 lb. and, assuming a coefficient of friction of 20%, there were 400 000 lb. of friction to resist the 240 000 lb. pull from the suspended panel point, *j*. As both spans were on rockers at Pier 4, the horizontal pull at this pier had to be transferred to the fixed end of Span 4 over Pier 5. Here, the reaction was comparatively small, so that, in order to prevent Span 4 from moving, under the action of this horizontal force, it was necessary to fasten the anchor end of Span 4. Fig. 20 shows the drop in the bottom-chord eye-bars at *j* immediately before closing Span 3.



SPAN 3 AT CENTER
BEFORE CLOSING

FIG. 29.

When everything was ready to close Span 3, traffic was interrupted at about 7 o'clock one morning and the track was broken. Jacks

were placed under jacking girders attached to the trusses at the shore ends of Spans 2 and 4 over Piers 2 and 5. As Span 4 had to move longitudinally about 6 in., the jacks under this span were placed on a carriage on rollers. These jacks were then pumped out. The top chords came together at *I*, then, as jacking was continued, the bottom chord straightened out, the hanger at *j* moved up until connection could be made at the top of the hanger, and the member, *Ki*, moved until it could be connected to the gusset-plate at *i*. Then, as the jacking proceeded, the stresses began to reverse, until finally all three spans were acting as simple spans, and there was zero stress in the erection ties. The D-plugs were then knocked out of the erection ties, the last plug being removed about 2 P. M., thus taking about 7 hours to swing Span 3. During all the jacking after the top chords had butted, Span 4 was moving north, the actual extent of this movement being about 8 in.

The total jacking was about 46 in. After Span 3 was swung, Spans 2 and 4 were lowered until they rested on the pier girders. Traffic was resumed about 6 P. M., after a delay of about 11 hours. This was the longest delay during the reconstruction of the bridge, and was the only serious one for the Railway Company. The entire closing was done without any serious difficulty.

I-beams were put on top of the new floor-beams of Span 3, and the old span was blocked up on these beams, and the old trusses and bracing were removed. The old trusses of Spans 2 and 4, which had been used for a counterweight, were also removed at this time.

5.—Change of New Floor to its Final Position.—This was the final stage of the erection. The large travelers were used to change the floor system, except a few panels at the shore ends of Spans 1 and 5, which could not be reached by the booms.

It had been planned to start simultaneously at each end of Span 3 and work toward the center, and, as traffic had to be stopped while a panel of floor was being changed, it was desired to change a panel with each traveler whenever an interval of time could be given. The wooden deck of the north traveler caught fire from the sparks of a train beneath, and as it was discovered late at night, the fire could not be put out quickly, and damaged the electric hoisting engine so badly that two steam engines had to be substituted for it. This

delayed the north traveler about 2 weeks, and made it impossible to start both travelers on the floor at the same time. The south traveler had changed the floor of the south half of Span 3 before the north traveler was repaired. From this time forward, however, when an interval was given to change the floor, each traveler would change a panel. On account of the heavy traffic, it was seldom that there was a chance to change more than one panel a day, and on some days no interval at all could be given. After the floors of Spans 2, 3, and 4 were changed, the erection ties over Piers 3 and 4 were taken down and re-erected over Piers 2 and 5. The travelers were then run back on Spans 1 and 5, and the floors changed with the travelers, except the five panels at the shore ends, which were changed by falls hooked to the new trusses.

The method used in changing a panel of floor was as follows: If the ends of the old stringers, in any panel of the span in which the change was to be made, did not clear the cover-plate of the new floor-beam by about 2 in., the old floor system was jacked on end until this condition was accomplished.

Two of the new stringers used in the material track were transferred to the inside of the trusses and hung from the truss by chains. The rivets connecting the old stringers to the old floor-beams were cut out and replaced by bolts, and those connecting the old stringer laterals to the stringers were cut out and replaced by one bolt in each connection. As many of the bolts fastening the new floor-beams to the hanger plates and the hanger plates to the bottom chord were removed as thought safe. Permanent lateral plates were fastened to the bottom of the new floor-beam, and the temporary lateral plates and angles on top of it were removed. The laterals at this point were supported by rope lashing attached to the bottom chord. A piece of rope lashing was run under all the ties in the panel to be changed. All this was done before the interval was given for the change.

As soon as the last train passed, rails were disconnected at each side of the panel, and this section was hauled on end to clear the panel. A line from the traveler was made fast to each end of the rope lashing under the old ties, and when the strain was taken on this line by the hoisting engine, all the ties in the panel were dumped

into the river below. These ties were in such bad shape that it did not pay to save them. Stringer laterals were removed, and the stringers were picked up with the booms of the traveler and outhauled back on the portion of the floor already changed, and piled at each side of the track.

A set of rope falls made fast to the sway bracing was now hooked to the old stringers in the next panel ahead, in order to hold them up after the old floor-beam was removed. As there was no convenient place to put the old floor-beam, it was hoisted up and hung by chains

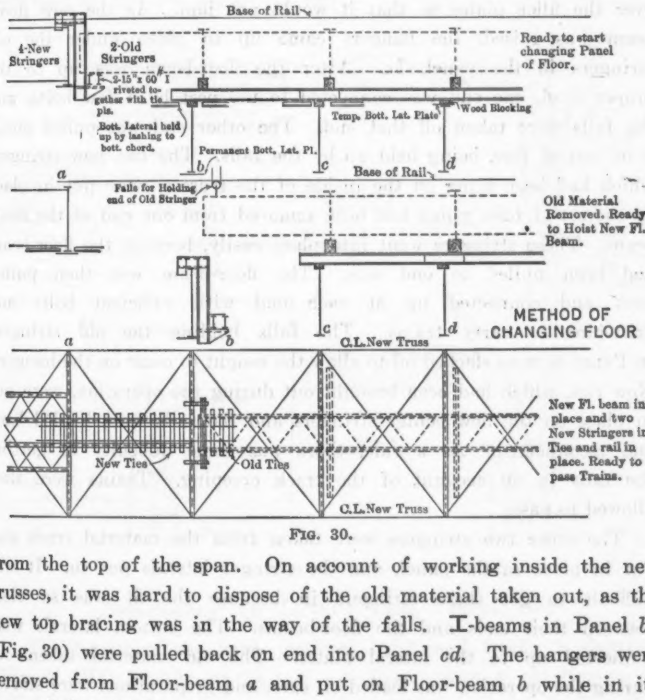


FIG. 30.

from the top of the span. On account of working inside the new trusses, it was hard to dispose of the old material taken out, as the new top bracing was in the way of the falls. I-beams in Panel *bc* (Fig. 30) were pulled back on end into Panel *cd*. The hangers were removed from Floor-beam *a* and put on Floor-beam *b* while in its suspended position. Falls from the traveler booms were then fastened to the ends of the new floor-beam to take the strain, and the remainder of the bolts connecting the floor-beam to the hanger plates and the hanger plates to the bottom chord were removed. The hanger plates were pulled out from between the floor-beam and the bottom chord

and the permanent filler plates were put in. These fillers extended above the top of the floor-beam in its final position, and were held in place by two bolts at the top of each plate.

Jacks, which had been placed between the new bottom chords ahead of the panel to be changed, were now worked, and the trusses were spread apart a little. The bottom laterals at this point had also been disconnected to allow the trusses to spread. The falls from the traveler boom were tightened, and the floor-beam, *b*, was hoisted to its permanent position. Men with bars guided the floor-beam over the filler plates so that it would not jam. As the new floor-beam was raised, the hangers came up in place under the old stringers in the panel, *bc*. After the floor-beam was up to the proper level, one end was connected to the post by a few bolts, and the falls were taken off that end. The other end was pulled about 3 in. out of line, being held up by the falls. The two new stringers which had been hung on the inside of the trusses, were put in place by the set of falls which had been removed from one end of the floor-beam. These stringers went into place easily, because the floor-beam had been pulled to one side. The floor-beam was then pulled back and connected up at each end with sufficient bolts and drift-pins to carry trains. The falls holding the old stringers in Panel *bc* were slacked off to allow the weight to come on the hangers. New ties, which had been brought out during the operation, were put in place on the new center stringers and the rails were hauled back and reconnected. As a rule, there was some difficulty in getting the rails in on account of the track creeping. Trains were then allowed to pass.

The other two stringers were taken from the material track and put in place in the panel, and the stringer laterals put in. It was difficult to get these stringers in because the fit was so close between their ends and the floor-beams. The bottom laterals were connected up to the lateral plates. The old material taken out during the operation was loaded on cars, and preparations were started for the change of the next panel. The average time that traffic was blocked during this operation was about 1½ hours, the minimum time being 55 min. It took a total of 7 weeks to change the whole floor, but this would have been cut down to 5 weeks, if the north traveler had not been delayed on account of the fire.

After the whole floor had been changed, ties were laid on the up-stream line of stringers, these ties being fitted in between those already laid on the center stringers. Track was laid, and the traffic was turned over to the up-stream track. The ties in the center were taken out and put on the down-stream line of stringers, enough extra ties being added to obtain the correct spacing. The center track was then thrown over, making a double track.

A 30-ton steel derrick with an 80-ft. boom was erected on Span 1 and removed the two south travelers; then it was erected on Span 5 and removed the two north travelers.

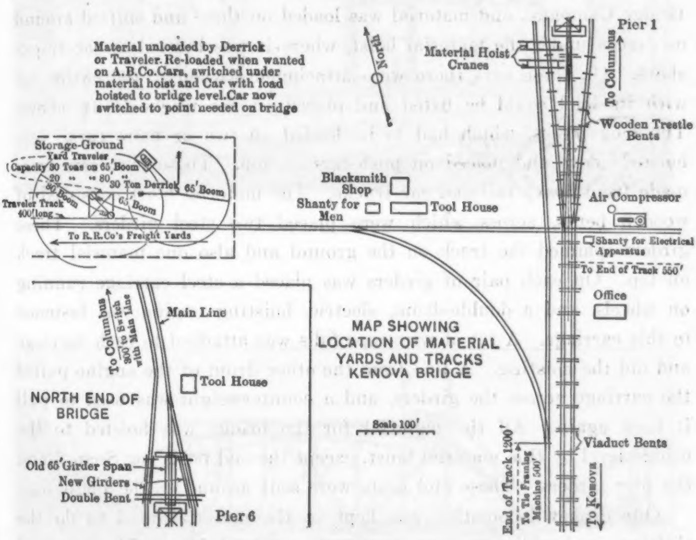


FIG. 31.

ARRANGEMENT OF MATERIAL YARD, TRACKS, ETC.

The layout of the yard, material hoist, and tracks is shown in Fig. 31. The location of the yard was down stream at the Kenova end of the bridge. It was equipped with one 30-ton derrick with a 65-ft. boom, the derrick being worked by an 85-h.p. steam engine. In addition to the derrick there was a steel traveler with an A-frame at each end having a 65-ft. boom on one end and an 80-ft. boom on the other. These booms were good for 30 and 20 tons, respectively. A track

for the traveler was laid on the ground, and this traveler handled all the lighter material. The chords for the 298-ft. spans were unloaded by the derrick; the 518-ft. span chords, each of which weighed more than 40 tons, were unloaded by using the derrick at one end and the traveler at the other. This traveler had previously been used in the erection of the Queen and Crescent Bridge at High Bridge, Ky. It was worked with two, 35-h.p., steam engines placed on its deck. The pier girders were unloaded by skidding them off cars and letting them rest on blocking.

There were seven flat cars on the job belonging to the American Bridge Company, and material was loaded on these and shifted around on tracks under the material hoist, where it was hoisted to the tracks above. On these cars there were attachments whereby the entire car with its load could be lifted and placed on the tracks 50 ft. above. The long pieces, which had to be loaded on two or more cars, were hoisted above and placed on push-cars on top. These push-cars were made from heavy railroad car trucks. The material hoist consisted of wooden bents, across which were placed two steel girders. These girders spanned the track on the ground and also one material track on top. On each pair of girders was placed a steel carriage running on wheels, and a double-drum, electric, hoisting engine was fastened to this carriage. A set of wire-rope falls was attached to each carriage and did the hoisting. A line from the other drum of the engine pulled the carriage across the girders, and a counterweight was used to pull it back again. All the material for the bridge was hoisted to the bridge level by this material hoist, except the end posts for Span 3 and the pier girders. These end posts were sent around to the main line.

One dinkey locomotive was kept in the material yard to do the shifting, and a lighter one was used on the bridge. The material tracks on the bridge had to be connected at each end with the main line. At the south end, this was done by a timber trestle; at the north end, the new 65-ft., deck, plate-girder span was used. A 30-ton, all-steel derrick-car with a 50-ft. boom, was kept on the job to move the gallows-frame from pier to pier and to handle the bottom of the end posts of Spans 1 and 5. After this work was completed, the derrick-car was sent away. A small wooden derrick-car of about 10 tons capacity was also kept on the site to handle light material, timber, etc.

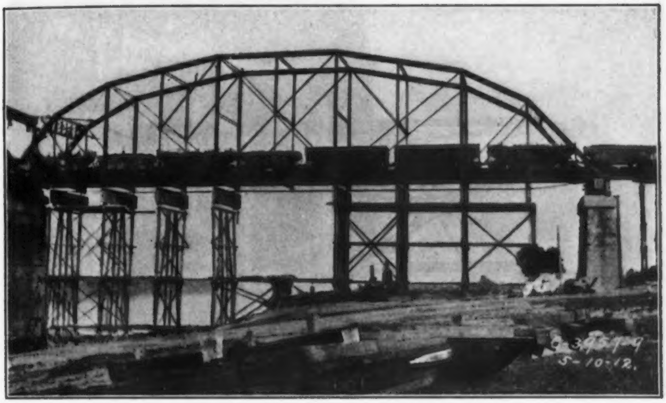


FIG. 32.—FALSEWORK UNDER SPAN 1.

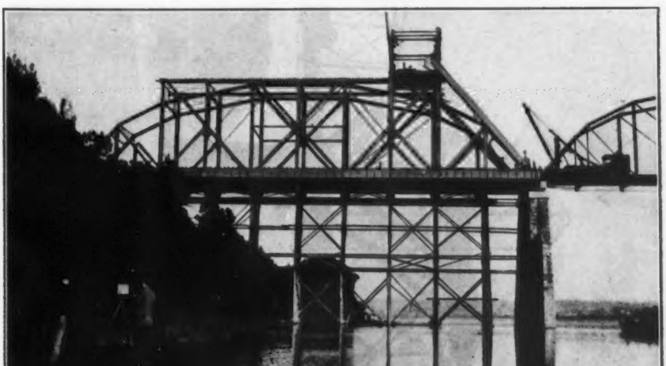


FIG. 33.—PLACING END POST OF SPAN 1.

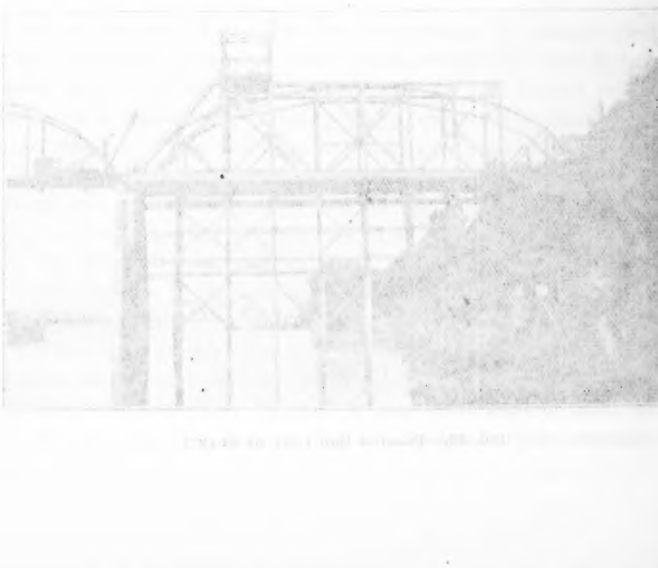


FIG. 34.—PORTALS OF OLD AND NEW SPANS 1.

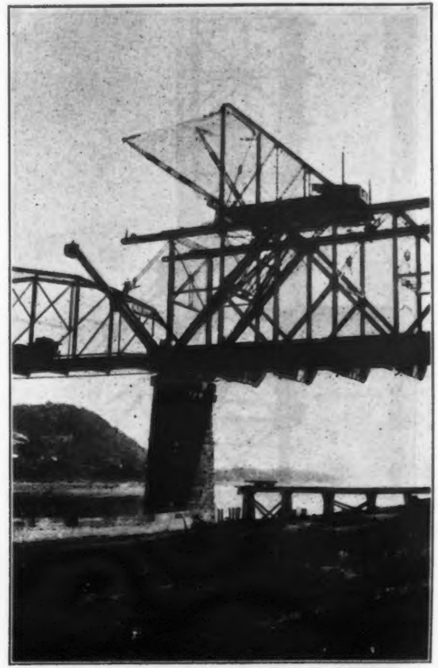


FIG. 35.—PLACING SHORE END POST OF SPAN 2.

FIG. 36.—MATERIAL HOIST, AND NEW PORTAL.

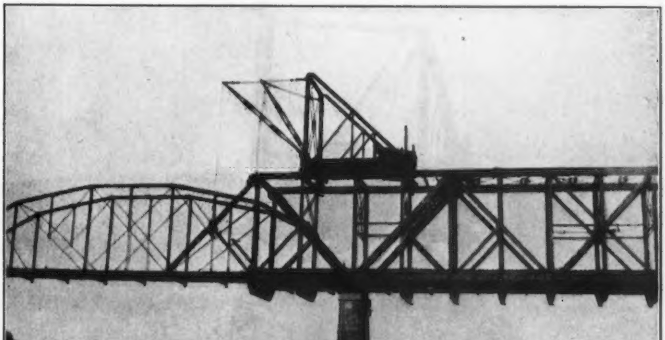


FIG. 37.—PLACING DIAGONAL OF SPAN 2.

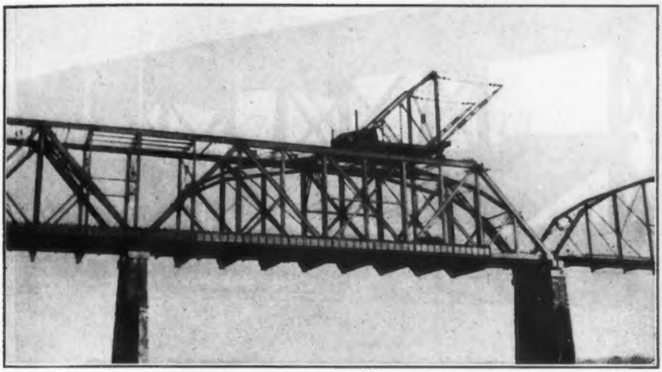


FIG. 38.—PLACING RIVER END POST OF SPAN 2.

FIG. 39.—LARGE TRAVELER CLIMBING SPAN 3.

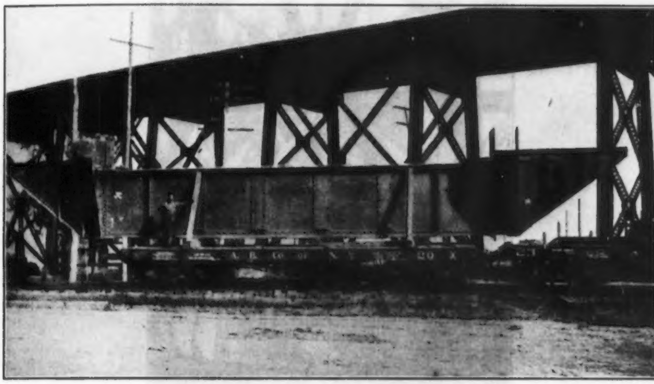


FIG. 40.—NEW FLOOR-BEAM AND TEMPORARY BRACKETS.

FIG. 41.—LIFTING NEW FLOOR-BEAMS INTO PLACE.

FIG. 42.—OLD AND NEW SPANS 3.

FIG. 43.—GAP AT TOP CHORD, SPAN 3.

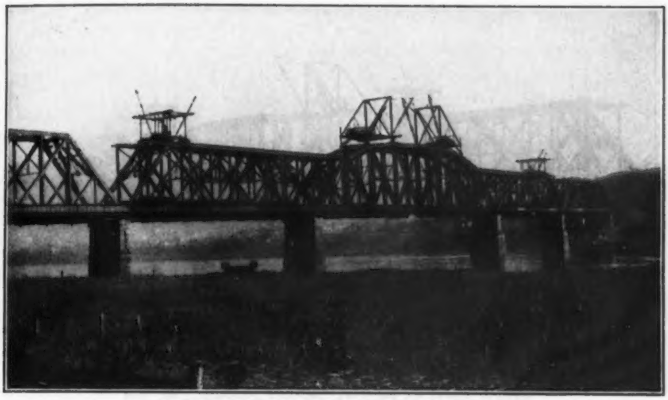


FIG. 44.—SPAN 3, BEFORE CLOSING.

FIG. 45.—JACK, ON ROLLERS, PIER 5.

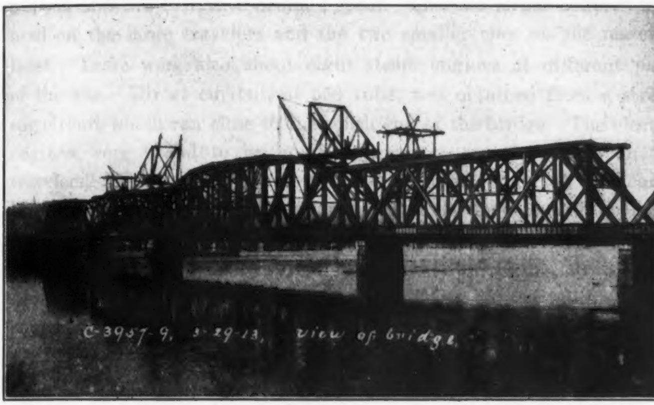


FIG. 46.—KENOVA BRIDGE, AT TIME OF HIGH WATER.

FIG. 47.—KENOVA, W. VA., DURING FLOOD.

There were four electric hoisting engines, two 100-h.p., 4-drum, 8-spool, and two 65-h.p., 2-drum, 4-spool. The two larger engines were used on the large travelers and the two smaller ones on the material hoist. There were also about eight steam engines at different parts of the site. Direct current, at 550 volts, was obtained from a street-car circuit which ran close to the south end of the bridge. The electric engines were found to be much superior to steam for use on the travelers, as there was no vibration from them when on the cantilever arm, the power was always there, and the trouble with coal and water was avoided. Rivets were driven by compressed air at about 100-lb. pressure furnished by a 12 by 12½ by 14-in., straight-line compressor with a capacity of 285 cu. ft. of free air per minute. This compressor was on the ground at the south end of the bridge.

The ties were framed by a tie-dapping machine run by a gasoline engine. This machine had a circular saw for squaring the ends of the ties, and on another shaft there were a number of small coarse-toothed saws to cut the dap. These saws were adjustable, so that a dap of any width could be cut, and there were levers to control the ties so that they would stop in the right positions for the various cuts. The ties were piled in large stacks, and the machine, mounted on four wheels, was moved from stack to stack. The ties were taken in at the front end, framed, and then piled back of the machine. It was found that by this method the framing cost much less than by hand. In removing the old spans, the oxy-acetylene torch was used to burn through the members, and this facilitated the work to a considerable degree.

Conclusion.

The new bridge is shown in Fig. 2. The weight of the approach viaduct is about 2 200 tons, and the weight of the finished river spans is about 9 600 tons. There were about 1 300 tons of erection material, such as the ties between the spans, the material track brackets, etc., making the total weight of steel about 13 100 tons. The entire reconstruction, covering a period of about 2 years, was carried on independently, first, of the heavy traffic over the bridge; and second, of the stage of the Ohio River. There were several floods during this period, the last being the great flood of March and April, 1913, which occurred about a month after the new Span 3 was swung. The water level rose about 65 ft., and the river practically covered the

Town of Kenova, but no damage was done which interfered with the railway traffic.

The general method of the reconstruction of this bridge was proposed by C. G. E. Larsson, M. Am. Soc. C. E., of the American Bridge Company. It was worked up by F. P. Witmer, M. Am. Soc. C. E., now Structural Engineer of the Brooklyn Rapid Transit Company, and Mr. William G. Grove, Engineers on designs, under the direction of C. W. Bryan, M. Am. Soc. C. E., Chief Engineer, and Mr. Larsson, Assistant Chief Engineer, of the American Bridge Company. The steelwork was fabricated at the Ambridge Plant. The erection was under the direction of Mr. J. B. Gemberling, Division Erection Manager. The work in the field was in charge of Mr. D. Burns, Foreman, and Henry Taylor, Assoc. M. Am. Soc. C. E., was the Field Engineer. For the Norfolk and Western Railway, the work was under the supervision of C. S. Churchill, M. Am. Soc. C. E., formerly Chief Engineer, and C. C. Wentworth, M. Am. Soc. C. E., Assistant Engineer. J. E. Crawford, M. Am. Soc. C. E., now Chief Engineer, was in charge of the alterations to the masonry piers and the checking of the steel superstructure, and F. P. Turner, Assoc. M. Am. Soc. C. E., now Bridge Engineer, was the Engineer in the field.

DISCUSSION

Mr. Cartlidge.

C. H. CARTLIDGE,* M. AM. SOC. C. E. (by letter).—The authors have presented, in a clear and comprehensive manner, a description of an extremely difficult and interesting work, and are to be congratulated on the success attending their carefully worked out details. Such descriptions in minute detail are valuable to those having similar problems, and are not frequently available. For the authors' presentation of the processes of erection and their elucidation of the details required for the particular erection procedure the writer has sincere admiration.

There are some matters concerning general design which seem to the writer to be worthy of inquiry. The steel selected for the spans, having an ultimate strength of from 62 000 to 70 000 lb. per sq. in., is, if similar to that furnished for some other and larger work fabricated by the same company, an extremely reliable and valuable material, the writer having been privileged to assist in some interesting tests of large columns of such steel. It would seem that the assumed loading is large enough to provide for any requirement, and that the unit stresses allowed should be as large as safety would permit. The process of designing for the ultimate possible load is, it may be stated parenthetically, the only logical one, and greatly to be commended. It is the practice of the writer in such cases. If it may be assumed that all the stresses which can come on the structure have been allowed for, it would seem that a material having a minimum yield point of, say, 34 000 lb. per sq. in., might be permitted a final or maximum working stress of, say, 80% of that, or about 27 000 lb. per sq. in. It is true that such working stresses are apt to produce somewhat of a shock to those who have so long been accustomed to thinking of stresses of 16 000 lb. per sq. in., but, with the postulates well in mind, the idea becomes reasonable. The unit stress of 24 000 lb. allowed by the designers for total stress, including, besides those due to live and dead loads, those due to wind and secondary stresses, seems somewhat conservative, and might well have been somewhat larger, say, 22 000 lb. for live, dead and wind load stresses, and not to exceed 26 000 lb. with secondary stresses.

The allowance for "dynamic effect" by the impact formula, $I = \frac{300}{300 + L}$, is the feature most open to criticism. For the chords of the long span, this gives an impact allowance of 36.6%, and in none of the many tests of long spans with which the writer is familiar, is there any evidence that such an impact effect is ever realized on the chord members of a span of this length. It is certain that for spans

* Chicago, Ill.

Mr. Cartlidge. of 500 ft. or more, the impact effect of a train running at the speed which will impose a maximum dynamic effect on the span is not much, if any, in excess of 10%, and this does not vary widely between webs and chords. The writer is not aware whether it is expected to run trains over such long spans, built at such a great height above the water, at speeds sufficient to produce an impact on the floor stringers of about 91 per cent. It would seem doubtful. Of course, it is not within the province of fair criticism to charge the designers with having provided too strong a structure. It is the right of the owners to provide as strong a bridge as they are willing to pay for, but the use of this particular impact formula leads to the overloading of some members with unnecessary material, the result being that the bridge is not of equal strength throughout. The design of the large compression members might have been improved had the walls been, as far as possible, of single thick plates. It is doubtful if any quantity of stitch riveting of two or more plates will provide a compression member of efficiency equal to one having equal sections without rivets. It is the belief of the writer, also, that for sections as large as these, the H-section is worth all it costs in somewhat more expensive details.

Mr. Thomson. T. KENNARD THOMSON,* M. AM. SOC. C. E.—This valuable paper is especially interesting to the speaker because he was Engineer of Bridges for the Ohio Extension of the Norfolk and Western Railroad, having supervision of the old Kenova Bridge and some 129 smaller ones, in 1890-1901.

William D. Janney, M. Am. Soc. C. E., of Baltimore, Md., was the Division Engineer at Kenova, and could add much interesting information relative to the foundations. These were all put in by the open coffer-dam method, there being practically no mud on top of the bedrock, which made it very difficult to get a water-tight joint, especially as there was no really low water, to speak of, in 1890.

The records for years back indicated that the water could be expected to drop to 6 ft. every summer, and in times of flood to rise to 66 ft. Those who know the Ohio River realize how sudden the rises are.

The coffer-dams were very well built, of 12 by 12-in. timbers, forming a double wall all around the proposed pier, with a space of about 5 ft. between the two walls, which was filled with earth. Much trouble was caused by leaks, although many scow loads of material were dumped around the outside.

On the west side of the river a couple of coffer-dams had been placed in 1890, but not unwatered that season. Next spring, when the water in the river fell, the coffer-dams were found to be filled to the top—25 ft. deep—with sediment.

* New York City.

One of the coffer-dams on the east side of the river had given especial trouble from leaks, and an attempt was made to seal it by dumping 5 ft. of concrete under water. The concrete set well, except around the edge, where the water still leaked in as badly as before. The surface of the concrete was also found to be very uneven. Mr. Thomson.

The erection of the steelwork was very rapid, as it was rushed on account of the danger of the falsework, 100 ft. high, being washed away. The dates for the erection, etc., are given in Table 3.

TABLE 3.—RECORD OF TIME OF ERECTION, ETC., OF THE OLD KENOVA BRIDGE, DURING 1891.

	Span 1.	Span 2.	Span 3.	Span 4.	Span 5.
Length of span, in feet	301	304	521	304	301
Erection of falsework started	July 30	Aug. 15	Sept. 14	Oct. 23	Nov. 4
" " " stopped	....	Aug. 17	....	....	....
" " " recommenced	....	Aug. 27	....	....	....
" " " finished	Aug. 8	Sept. 6	Sept. 28	Oct. 30	Nov. 12
Erection of traveler started	Aug. 10	....	Oct. 2	....	....
" " " finished	Aug. 13	....	Oct. 6	....	....
Erection of ironwork started	Aug. 18	Sept. 8	Oct. 6	Oct. 31	Nov. 12
Ironwork finished and span swung	Aug. 25	Sept. 12	Oct. 21	Nov. 4	Nov. 16

Erecting a 300-ft. span in 3½ days was by no means a slow job. It was accomplished by the Baird Brothers, well known for their rapid work, and also for their recklessness.

In preparing for rapid erection, a little too much clearance was allowed between the stringers and floor-beams, with the result that, when the spans were swung free of the falsework, about thirty or forty of the connecting angles were cracked at the fillets and had to be cut out.

The four end piers were completed in 1890, and the three main river piers in 1891, the 100-ft. piers being built in less than 2 months each.

The actual costs of the substructures of this bridge were:

Masonry piers and abutments	$165 285
Viaduct pedestals	12 020
Shore foundations	6 932
River foundations	128 157
Inspection	4 288
Total cost	$316 682

The calculated weights of the superstructure were:

Viaduct	1 519 500 lb.
Four 298-ft. spans	2 734 000 "
One 518-ft. span	2 146 500 "
Total weight	6 400 000 lb.

Mr. Skinner. F. W. SKINNER,* M. AM. SOC. C. E.—Mr. Thomson's reference to the Baird Brothers brings reminiscences of those old-time bridge erectors who played a major part in the development of the art of American bridge building which has far surpassed that of all other nations. Bridge erection has not only kept pace with the design and fabrication of long-span bridges, but it has anticipated it, and by providing new methods, creating new apparatus, and training experts, has led the way and indicated the lines on which these great structures have progressed with a safety, speed, and economy unapproached abroad.

The art of bridge erection has been wholly and entirely created within the last 40 years, and some of its masters are yet winning great victories and establishing new records. When, less than 30 years ago, steel spans inaugurated the construction of trusses more than 550 ft. long, both plant and methods were simple and crude, and no special facilities existed, the multiple-drum hoisting engine was unknown, as was the pneumatic hammer, and very little power was available for field work, even on the largest jobs. Electric power was unheard of, and large hydraulic jacks were not on the market. Derricks and tackles seldom or never had capacities of more than 30 tons, wire-rope tackle was rarely used, and single pieces or truss members did not exceed 30 or 40 tons weight and could not have been transported if they had.

The erectors, however, never hesitated to swing long and lofty spans over torrent, chasm, and flood, and, working with much greater hardship and peril than now, succeeded in the face of great disadvantages. Many a time the erection was a race for life with ice, storm, flood, and scour, and the erectors were real men with splendid courage, loyalty, zeal, resourcefulness, and fidelity, ready for every emergency and anticipating the dangers and difficulties coolly, successfully, and modestly. Among those whom it has been the speaker's privilege to know best are some of the greatest of all, such men as William, Robert, Andrew, and John Baird, Robert Grimes, Milton Clay, the late H. F. Lofland, M. Am. Soc. C. E., H. A. Green, J. B. Gemberling, W. H. Wilkinson, John Watson, John Devin, L. N. Gross, Newt. Jarrett, Augustus Milliken, S. P. Mitchell, M. Am. Soc. C. E., Phelps Johnson, M. Am. Soc. C. E., and the engineers of the railroads and bridge companies. To them, to their comrades and disciples, some of whom are still directing the greatest constructions yet attempted, is due a large share of the credit for our most splendid bridges.

Mr. Jewel. L. L. JEWEL,† M. AM. SOC. C. E. (by letter).—There is perhaps no field of engineering endeavor in which boldness, tempered with sound judgment, reaps such rich rewards as in the erection of structural steel. This quality of intelligent daring is particularly demanded

* New York City.

† Saranac Lake, N. Y.

in the erection of heavy modern bridges, the size and weight of their members alone making their transportation and assembly a difficult problem. When to this is added the uncertain action of a treacherous stream, and the necessity of maintaining clear paths for traffic both above and below, conditions are created which demand a very thorough knowledge of engineering principles and a high order of executive ability; and these, perhaps, are not always fully appreciated by the purely designing engineer. The authors have given a very clear and concise description of such an undertaking, and of the methods by which it was brought to a successful completion. Mr. Jewel.

The study of such a paper or problem will be along four lines of primary importance, namely, risk, cost, time, and interference with other operations. As all these considerations are largely interdependent and matters of judgment rather than fact, there is generally ample room for wide difference of opinion. The case described by the authors is not an exception to this general rule in regard to certain parts of the work.

It is unlikely that any one familiar with erection costs will disagree with the authors' statement that the two shore spans, Nos. 1 and 5, could have been erected at less cost by the method of straight falsework and a gantry traveler; nor does it seem open to question that this cheaper method would have also taken less time, and, to that extent at least, would have lessened interference with the railroad traffic. There is, however, some question in the writer's mind whether this cheaper method would not have really involved less risk—the most important consideration of all—than the method used.

The deflection of these two spans under their regular traffic load is given in Fig. 28, as $\frac{3}{8}$ in. and $1\frac{1}{2}$ in. below the dead-load camber for Spans 1 and 5, respectively, the great difference being due no doubt to excessive wear in the pin-holes of Span 5. The maximum deflection of Span 1 under its erection load is given by the authors as "about $2\frac{1}{2}$ in. below the dead-load camber, or $\frac{1}{2}$ in. below a horizontal line drawn from center to center of the end pins", which is a little more than is shown in the theoretical diagram, Fig. 22. A comparison of the erection-load and the live-load deflections for Span 1 is $2\frac{1}{2}$ in. to $\frac{7}{8}$ in., or nearly 3 to 1. At this point in the process of loading the old span with the new one, no top chords or end posts had been placed, and, moreover, the old span had been relieved of the weight of its own track and track stringers.

It would seem, therefore, that with the falsework carrying the old track stringers and track, and also a more or less indeterminate part of the weight of the new work, the old span was loaded very nearly to the line dividing safety from risk. It was carrying a uniform load of about 7 000 lb. per lin. ft. of new steel, the weight of two material tracks, the brackets supporting them, the erection blocking.

Mr. Jewel. hangers, etc., and the traveler track and its falsework—a uniform load approximating 8 500 lb. per lin. ft. In addition to this uniform "dead" load, it carried the erection traveler and the weight of the heavy chords, while being placed, concentrated at two panel points. Such a traveler hoisting, swinging, lowering, and suddenly stopping heavy loads is certainly the liveliest kind of a "live" load, and, on occasion, its total weight may be carried largely by only one truss.

Taken altogether, this was certainly a very severe load for a span designed for Cooper's E-40, 25 years ago, and in hard service for 21 years. If, during erection, a rise in the river should have destroyed the falsework—which was merely an auxiliary to erection—an addition of about 2 000 lb. per lin. ft. would have been made to this already heavy load. However, the greater part of this addition, that is, the weight of the top chord, would have extended only from one end up to, and including, the position of the traveler.

Additional value would have been given to an already valuable and purely erection paper, if the authors had stated the assumed loading, including traveler concentrations, for these two old spans, and the unit stresses involved, instead of, or in addition to, the stress sheets for the new spans. It is, of course, very desirable to be far more liberal as to unit stresses for a temporary purpose, such as the comparatively short period of erection, than for a permanent structure of the same kind, but there is always the question as to just how far this liberality should go. The writer has allowed a stress practically equal to the elastic limit of the material in certain heavy erection equipment, but only when such stress could occur a very limited number of times, at long intervals, and for only a few minutes at a time. Moreover, the material was new and subjected to the same rigid inspection and tests as the material for the permanent structure. Judging merely from a comparison of the deflections given, a reasonable assumption of the traveler's weight, and the loading for which these old spans were designed, and without making any calculations, it would appear that such a stress was exceeded perhaps in the present case.

As to the risk involved in depending on falsework for the entire erection load as well as for traffic, some doubt may be expressed whether there has ever been a flood in the Ohio River, barring ice, which would have destroyed well-constructed falsework, of the type necessary for this erection, under the two shore spans. At any ordinary stage of water, all of Span 1 is practically over dry land, and at no stage is there a current under this span which could be considered very dangerous, either from its direct force or scour—a condition due partly to the wide expanse of low flat land on the eastern, or rather southern, side which allows extensive spreading and also, of course, to the distance of Span 1 from the line of principal flow. Span 5

does not have so much advantage of location, but it is also some distance from the center of the channel and well out of the maximum current at any stage of the water. Ice can hardly be the serious consideration here that it is farther north (in the Pittsburgh District), and, in any event, it goes out in the spring for all the rest of the year. Mr. Jewel.

In expressing a preference for the straight falsework method over the one used for the two shore spans, the writer does not forget that "nothing succeeds like success" and that the method used has been proved out. At the same time, he believes that many of our larger rivers bear a blanket reputation for deviltry without any consideration of the fact that every stream's character changes very materially with the topography of its valley. Many of our "bad" rivers are also charged with numerous wrecks due to absolutely avoidable conditions or causes. Every one interested in erection work has observed the use of falsework, for example, which could be more accurately classified as a gambling device with the chances against the contractor, than as an engineering structure. Without reference to the case under discussion, it may be said that every erection enterprise deserves individual study and decision, unprejudiced by records of disaster at other locations.

Another point which seems worth mentioning for comparison is an alternate method for delivering material for the three spans erected by the cantilever method. Apparently, there was sufficient depth of water for a barge, because all floor-beams were lowered from the bridge to a barge and turned so as to be hoisted by lines on each side of the bridge. The question naturally arises, why was not all material delivered directly from yard to traveler by barge and tugboat? A considerable percentage of the 1 300 tons of special erection material would have been saved by omitting the material track brackets, as well as considerable labor in building and dismantling the tracks. By the barge method, all new truss material would have had one less, and all new floor material two less, handlings; and it would appear from the photographs that almost any stage of water which would hinder the operation of the tugboat would flood the material yard and so stop operations in any event. This alternate method is suggested only on the assumption that there could never be lack of water to float the barge under the traveler.

The cantilever portion of the work is very interesting. Although it shows nothing entirely new, it is the heaviest simple truss work erected by this method that has come under the writer's observation, and he would like to express appreciation of the great mass of details which required accurate working out in order that all clearances and the closure of the channel span could be so well provided for. The authors' statement that the 518-ft. trusses are the largest riveted trusses ever built, may be the case if over-all dimensions only are con-

Mr. Jewel. sidered. Much heavier simple truss spans, which are riveted throughout, are the three channel spans of the Union Depot bridge and Terminal Company's bridge over the Missouri River, at Kansas City. One span, which carries a lifting deck to give clearance for navigation, is 425 ft. 6 in. from center to center of end bearings, and contains 4 823 tons of steel, including machinery. The two fixed spans are 423 ft. 0 in. from center to center of end bearings. Each contains 4 008 tons of steel, about $9\frac{1}{2}$ tons per lin. ft. They are double-deck spans carrying a double-track railroad below, and street-car lines, carriageways, and sidewalks above. The bridge was designed by Waddell and Harrington, Consulting Engineers, and was fabricated and erected by the McClintic Marshall Construction Company, Paul L. Wölfel, M. Am. Soc. C. E., Chief Engineer. The writer was, at that time, Manager of Erection for the contractors, and was responsible for the erection plans and methods. The field work was completed under the management of E. A. Gibbs, Assoc. M. Am. Soc. C. E., and W. R. Hughes, Jr., Assoc. M. Am. Soc. C. E., was Resident Engineer.

The erection presented no unusual features except the great loads involved, several members in each span weighing considerably more than 100 tons, and the fact that a wedge device operated by a screw was used under each panel point in place of the usual crude wooden wedges. In this manner, it was not difficult to assemble the bottom chords in a level line, and thus all the chord splices, which had been assembled previously and drilled, were made practically perfect. After they were riveted, the wedges were lowered the proper distance at each panel point, working from the center to each end, and the massive chords were quickly, easily, and accurately brought to the position of no stress. These same wedge devices were used by the writer to lower the Panama lock gates over their pintles.

Messrs. Grove and Taylor. WILLIAM G. GROVE,* ESQ., and HENRY TAYLOR,† ASSOC. M. AM. SOC. C. E. (by letter).—The writers have read with much interest the discussions by Messrs. C. H. Cartlidge and L. L. Jewell, and some explanation of a few of the points raised therein, would seem to be necessary before the matter is closed.

At present, the live load on structures is limited by the capacity of the rails, but it is probable that the future will show an improved grade of rail which will allow the use of heavier motive power and rolling stock. The Norfolk and Western Railway Company, therefore, with this idea in view, wished to provide a bridge which would be capable of giving service for a long time, and hence felt entirely satisfied with the quantity of steel put into the new structure.

* New York City.

† Philadelphia, Pa.

In regard to the proper impact formula to be used in the design of railroad bridges, there is a great diversity of opinion among engineers. Without attempting to defend the particular impact formula adopted in the design of the Kenova Bridge, which formula is now used by the great majority of the railroads of the United States, and after twenty-five years of application, has been found to give satisfactory bridges, the writers are of the opinion that Mr. Cartlidge has not correctly applied the formula in the illustration he quotes, as the L in that formula is the length of loaded track. As this bridge is a double-track structure, the loaded length for the chords of the 518-ft. span is 1 036 ft., giving an impact allowance of only 22.4%, instead of 36.6%, as mentioned in Mr. Cartlidge's discussion. Messrs. Grove and Taylor.

The stresses in the truss members of old Spans 1 and 5 were very carefully worked out in advance for all possible conditions of loading these spans during the erection of new ones. The worst conditions of loading gave a maximum unit stress of about 23 000 lb. per sq. in. in one or two of the bottom chord eye-bars, and this only for a short time. Although, no doubt, these old spans were subjected to greater strains under the weight of the new span than from any train load that was put on them, it is also true that the travelers used were, from their construction, rather slow machines, and were equipped with electric engines, which gave very little vibration when running. This, however, is not the case with train loads repeated many times during the day. From observation during the work, the writers cannot agree with the statement that the traveler was "the liveliest kind of a 'live' load", and the judgment of the engineers, that old Spans 1 and 5 would safely support the new spans, was borne out in the construction of the work.

It is certain in the minds of the writers that any falsework which might have been used to erect new Spans 1 and 5 would have been washed out in the flood which occurred during the progress of the work, had it been supporting the new steel at that time. There was a rise of 65 ft. from low water, and a very considerable current under Spans 1 and 5. Many large houses, and wreckage of all kinds, passed under the bridge for several days, and it is hard to believe that any falsework of the ordinary type could have stood the pressure which would have been developed against it.

The method suggested of bringing out all the new material on barges was considered, of course, when the plans for the erection of the bridge were being worked up. During that summer (1911), there was not enough water under the greater portion of the bridge to float a barge, and this condition lasted for some months. It was decided, therefore, not to depend on the barge method, although it happened that during the actual reconstruction of the bridge, for the greater

Messrs. Grove and Taylor.

part of the time, there was sufficient depth of water to have used barges. From previous investigations of records covering a long period of years, however, this was very unusual.

In conclusion, the writers can safely state that both the Norfolk and Western Railway Company and the American Bridge Company feel entirely satisfied with the design of the bridge and the general method of reconstruction.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

TRANSACTIONS

Paper No. 1340

THE ST. JOHN LEVEE AND DRAINAGE DISTRICT OF MISSOURI*

By R. M. Strohl, Jun. Am. Soc. C. E.†

With Discussion by Messrs. Vernon M. Eager, J. S. Spiker, and R. M. Strohl.

Synopsis.

In view of the fact that, at present, there is little information relating to the quantity of run-off, impounded storage, back-water curves, and other matters of direct importance to drainage districts in the Mississippi Valley, and that even these vary greatly in different localities, it is believed that a record of such features for Southeastern Missouri will be of some value; also, as little has been written concerning the assessment of benefits accruing from the construction of reclamation systems, an outline of this work, as applied to the territory included within this district, will be of value to those engaged in operations of this kind.

A record of the assumptions and data on which the work of the St. John Levee and Drainage District has been designed, compared with the actual results obtained after its completion, will give an authoritative basis for future undertakings.

This paper treats of some of the drainage problems of Southeastern Missouri, and gives in detail the data necessary to be considered in designing a system for the complete and economical reclamation of

* Presented at the meeting of April 7th, 1915.

† Now Assoc. M. Am. Soc. C. E.

large tracts of overflow land. Levees, floodways, drainage ditches, floodgates, storage basins, and siphons, are considered and discussed in detail as they are found necessary.

The assessment of benefits, as provided for under the Missouri laws, is also outlined and applied to the case, and an effort has been made to show a just and equitable distribution of the cost of the proposed work.

Location.—The St. John Levee and Drainage District of Missouri, Fig. 1, is in that part of Southeastern Missouri lying along the Mississippi River and across from Cairo, Ill. It is bounded on the north by Scott County and the Big Lake Drainage and Levee District of Missouri, on the east and south by the Mississippi River, and on the west by a low ridge known as the Sikeston Ridge. The southern portion of this ridge is included.

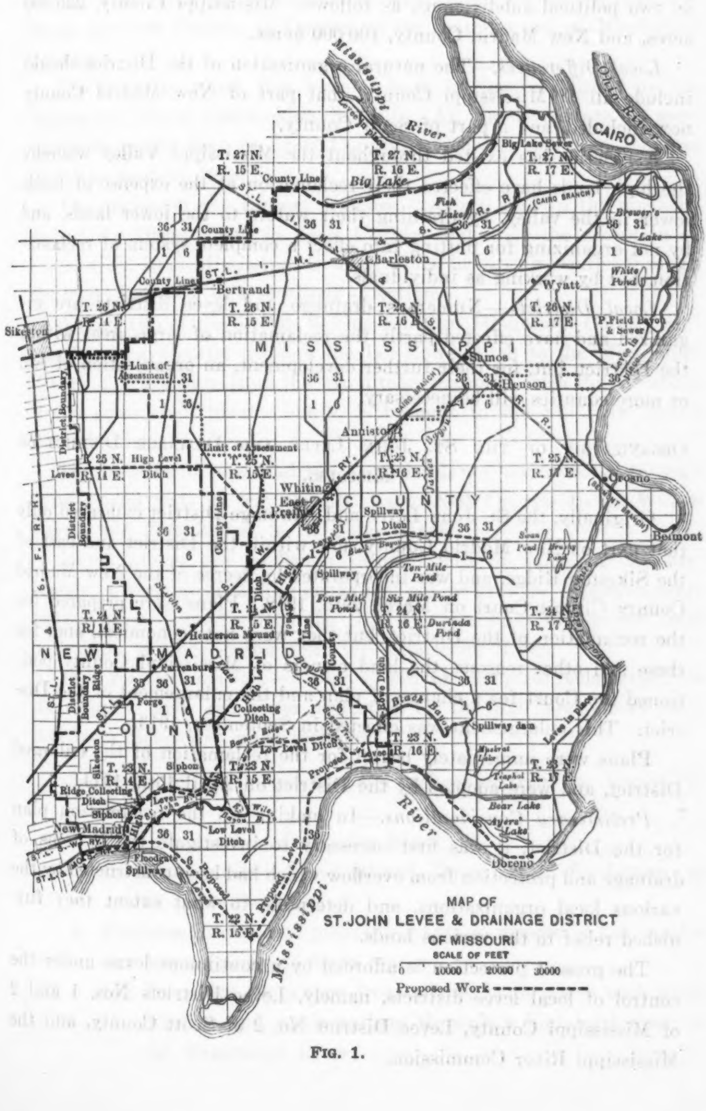
Soil.—The lands of the District are especially fertile, having been built up by deposition by the Mississippi River. The soil in the northern portion of the District consists largely of a sandy loam which is easily cultivated. Underlying this at no great depth is a stratum of sand. As the southern end of the District is approached, the sand stratum is found at an increasing depth, and is overlaid by finer sedimentary deposits, gumbo and clay, varying in thickness from 2 to 20 ft., or more.

Surface Features.—The surface is generally a plain with an average slope to the south of about 1 ft. per mile.

Two natural basins, locally known as Ten Mile Pond Basin and Eagle Nest Cypress Basin, are found in the southern part, and are separated by Sugar Tree Ridge, which extends northwest and southeast to a point about ¾ mile from the Mississippi River at the south central part of the District.

Natural levees have been forming and, except for two bayous, namely, the St. John Bayou and James Bayou, have shut off effectively the drainage from the interior.

There are three outlets, the two bayous just mentioned and one artificial channel, for the drainage of the interior lands. Several small bayous, namely, East, Wilson, and Black Bayous, aid in the drainage of the lands in the southern part of the District.


Mississippi River
Ohio River
CAIRO
Big Lake Sewer
Big Lake
County Line
Fish Lake
Brewer's Lake
White Pond
Charleston
Bertrand
Wyatt
Sikeston
P. Field Bayou & Sewer
MISSISSIPPI
Samos
Henson
Limit of Assessment
District Boundary
Anniston
High Level Ditch
Levee
Whiting
East Prairie
COUNTY
Spillway Ditch
Swan Pond
Ten Mile Pond
Spillway
Four Mile Pond
Six Mile Pond
Duvalls Pond
Belmont
Crosno
Henderson Mound
NEW MADRID
Farrenburg
St. John Bayou
Collecting Ditch
Low Level Ditch
Black Bayou
Spillway dam
Siphon
Ridge Collecting Ditch
New Madrid
Floodgate
Proposed Levee
Doreno
Bear Lake
Ayers Lake
Teaphol Lake
Muskrat Lake
T. 27 N. R. 15 E.
T. 27 N. R. 16 E.
T. 27 N. R. 17 E.
T. 26 N. R. 14 E.
T. 26 N. R. 15 E.
T. 26 N. R. 16 E.
T. 26 N. R. 17 E.
T. 25 N. R. 14 E.
T. 25 N. R. 15 E.
T. 25 N. R. 16 E.
T. 25 N. R. 17 E.
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T. 23 N. R. 16 E.
T. 23 N. R. 17 E.
T. 22 N. R. 15 E.
MAP OF
ST. JOHN LEVEE & DRAINAGE DISTRICT
OF MISSOURI
SCALE OF FEET
0 10000 20000 30000
Proposed Work

FIG. 1.

Area.—The District now embraces about 325 000 acres, taken up by two political subdivisions, as follows: Mississippi County, 225 000 acres, and New Madrid County, 100 000 acres.

Local Differences.—The natural organization of the District should include all of Mississippi County, that part of New Madrid County now included, and a part of Scott County.

A practice has existed throughout the Mississippi Valley whereby the high lands have effected their reclamation at the expense of lands lower in the valley, by draining their waters to the lower lands, and by not organizing for putting into effect a complete system of reclamation, but by working as individuals.

Local Districts.—Numerous drainage and levee districts are organized and have effected partly the reclamation of large areas within the District, but, for their further development, an organization of two or more counties will be necessary.

Organization of the St. John Levee and Drainage District of Missouri.

Originally, the St. John Levee and Drainage District embraced only that part of New Madrid County now within the District and east of the Sikeston Ridge, and was incorporated by decree of the New Madrid County Circuit Court on March 29th, 1912. Plans were prepared for the reclamation of the District, but they were not economical, and, for these and other reasons, the land owners of Mississippi County petitioned the Court for a change of plan and the enlargement of the District. This enlargement was effected in September, 1913.

Plans were immediately begun for the reclamation of the enlarged District, and were adopted by the District on April 15th, 1914.

Preliminary Considerations.—In making up the reclamation plan for the District, it was first necessary to investigate the systems of drainage and protection from overflow which had been constructed by the various local organizations, and determine to what extent they furnished relief to the various lands.

The present protection is afforded by a continuous levee under the control of local levee districts, namely, Levee Districts Nos. 1 and 2 of Mississippi County, Levee District No. 2 of Scott County, and the Mississippi River Commission.

This levee varies from 5 to 18 ft. in height, and has a total length of about 55 miles; it extends from what is known as Philadelphia Point, in Scott County, to Dorena, at the southern end of Mississippi County.

The grade to which this levee is built is that established by the Mississippi River Commission in 1910. All the levees are in good condition except that in Scott County, which has not been repaired since the flood of 1913.

The ditches of the various drainage districts head in the northern end of the District and in Scott County, and, with the exception of District No. 30 of Mississippi County, which has its own outlet through the levee, they flow in a southerly direction.

All but two of the ditches flowing south in Mississippi County converge and have outlet through the Lee-Rowe Ditch to the Mississippi River. The drainage from the two ditches excepted, with that of about 90 000 acres from Scott County, is brought to the St. John Bayou through dredge ditches.

The drainage of New Madrid County lands not having outlet through these ditches finds its way through Eagle Nest Cypress Basin, Wilson Bayou, and East Bayou, to St. John Bayou.

Many ditches are inefficient through lack of sufficient capacity, largely on account of sliding banks and willow growth. Approximately 187 000 acres of land within the District are subject to overflow from back-water and head-water from the Mississippi River.

Plan for Reclamation.—A plan to provide complete reclamation for the overflow lands and better protection for the high lands above overflow from back-water, must include:

1. Levees to prevent overflow and back-water from the Mississippi River;
   (*a*) Enlargement of existing levees;
   (*b*) Construction of additional levees with flood-gates at St. John Bayou;
2. Drainage ditches and pumping plants to dispose of the surplus rain-water falling on these lands;
   (*a*) To collect the waters from all the lands above back-water overflow and conduct them through a high-level ditch to the Mississippi River;

(b) To conduct through a low-level diversion ditch the remainder of the surface water which would flow out at James Bayou and the Lee-Rowe Ditch, to St. John Bayou, so as to permit the closing of these outlets, and thus lower the head of the back-water;

(c) Collection ditches on the lands below overflow and a pumping plant to take care of the surplus water during periods of overflow in the Mississippi River or when the flood-gates must be closed.

Conditions can be greatly improved in two or three of the existing ditches. This, however, will not be taken up in this plan, but will be left for improvement as the land owners along each ditch may desire.

District Boundaries.—It requires only a casual examination to disclose the fact that in order to work out a complete and economical project, all the territory south of the Commerce Hills and between the Sikeston Ridge and the Mississippi River must be considered; and the District should be enlarged to include all territory not now within the boundaries of the St. John Levee and Drainage District of Missouri.

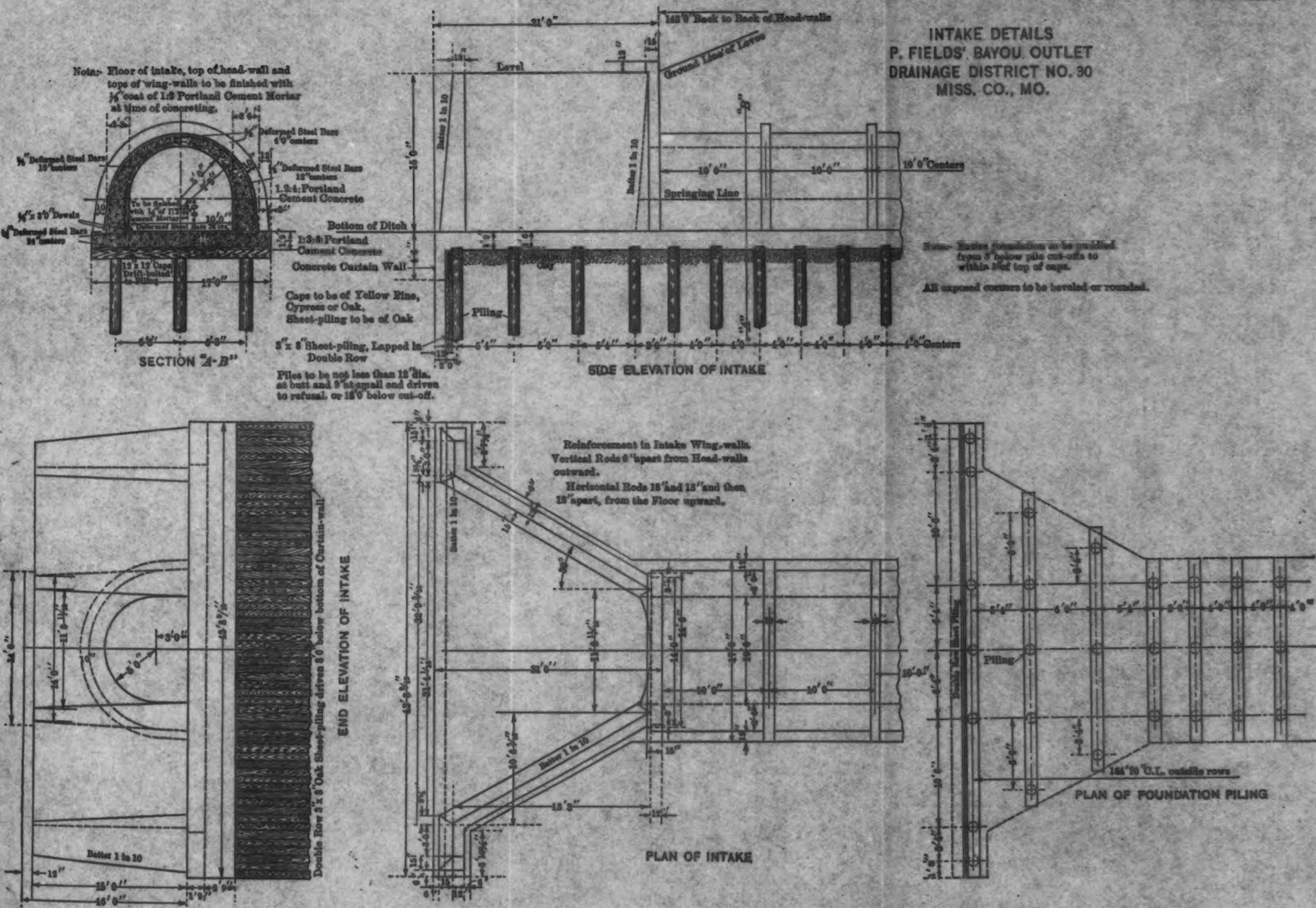
In addition, the proposed improvements will affect materially that portion of the Little River Valley which is subject to severe overflow from waters that pass across the Sikeston Ridge in the vicinity of New Madrid; and the District should be enlarged to include the lands lying between the St. John Levee and Drainage District and the Little River Drainage District.

Levee Protection.—As nearly all the lands within the District and those which should be included are subject to overflow by the Mississippi River, it is necessary to provide a levee sufficiently high and strong to confine these waters to their natural channel.

Investigations were made to determine the elevation of the flood planes of the various overflows. Since the overflow of 1913, it has been necessary to provide protection against floods of equal volume and height, and as the Mississippi River Commission grade of 1910 was no longer considered safe, a new grade was decided on for this work, and, later, a grade was established by the Mississippi River Commission, which would be of sufficient height to confine the volume of the flood of 1912.

PLATE X.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXIX, No. 1340.
STROHL ON
DRAINAGE PROBLEMS
IN THE MISSISSIPPI VALLEY.

INTAKE DETAILS
P. FIELDS' BAYOU OUTLET
DRAINAGE DISTRICT NO. 30
MISS. CO., MO.



The junction of the Ohio and Mississippi Rivers brings about a difficult condition, and, at times of flood, the effect of their confluence can be readily seen. Fixing a grade at from 3 to 4 ft. above the 1913 flood-plane calls for an unbroken levee from Commerce to New Madrid. At present there is no levee from Commerce to Philadelphia Point in Scott County, nor from Dorena, in Mississippi County, to the high land above the overflow about 1 mile below New Madrid.

The existing levee is to be used where it can be enlarged satisfactorily. This enlargement can be effected on all except about 1½ miles of poorly located levee in Scott County. The levee will be brought to the standard of the Mississippi River Commission throughout, namely, an 8-ft. crown with 3 : 1 side slopes and a 4-ft. muck-ditch, which it has fixed for this river district.

At the crossing of the levee and James Bayou and the Lee-Rowe Ditch, the section will be increased to an 8-ft. crown with 4 : 1 side slopes. A 40-ft. banquette, beginning 10 ft. below the crown, is to be added on the inside. One row of sheet-piling will be placed near the center of the levee in order to cut off seepage.

A special levee has been designed for the water-front at New Madrid. To protect the property of the city, the levee must be as close to the river as possible, where the bank is protected by revetment placed about 15 years ago. In order that the property damage might be limited and few removals of buildings be made necessary, a broad, flat levee with a crown width of 10 ft. and 10 : 1 side slopes was decided on. Buildings and dwellings on the site are to be raised on their present locations above levee grade, and the material for the levee is to be pumped in from the river by a suction dredge.

Drainage Outlets Through the Levee.—There will be three openings through the levee, two of which have already been provided for by local organizations, namely, the Big Lake Drainage and Levee District of Missouri and District No. 30 of Mississippi County, Missouri, the third being the outlet of St. John Bayou under this plan.

Design of Outlets.—The accompanying drawings, Fig. 2 and Plates X to XIII, show the general plan of outlet, as designed by the writer, for District No. 30. This outlet is placed in what is known as P. Field's Bayou. The gate and hoist have been furnished by the Coffin Valve Company.

Design of St. John Bayou Outlet.—Plate XIV shows the general plan of the St. John Bayou Outlet. Details of the gates, gate-frames, and hoists will be in conformity with the standards of the manufacturers who furnish them.

Foundation conditions for the St. John Bayou Outlet, as shown by borings, will necessitate strong coffer-dam construction, steel sheet-piling being deemed best for the work. An open coffer-dam with inside dimensions of 110 by 150 ft., and piling from 24 to 30 ft. in length, will be required. It will also be desirable to have the coffer-dam free from bracing, in order to allow the work to proceed without obstruction, and a plan to this effect has been developed.

Drainage.—Practically all the run-off from the lands of the District, as well as from about 90 000 acres in Scott County, passes into the Mississippi River through the three outlets previously mentioned.

The cutting of the Lee-Rowe Ditch directly into the Mississippi River was ill-advised, as an efficient outlet could have been secured through James Bayou at less cost, and it would not now be necessary to close this opening.

Of the 415 000 acres draining into the Mississippi River, about 190 000 acres are still subject to overflow from back-water from the river at flood stages like that of 1913. This leaves about 225 000 acres that will drain by gravity to the river at a stage of 44.6 ft. on the New Madrid gauge, which was the high-water mark of the 1913 flood and was never before equalled; it corresponds to an elevation of about 307.19 ft., Memphis datum, to which all work in this district is referred.

With the building of the levee along the river and the consequent closing of the drainage outlets, it is seen that unless the water from the lands above back-water overflow is provided for separately, it will be a source of damage and a burden to the lands subject to such overflow.

To meet this, an intercepting or diversion floodway, or high-level ditch, was planned. This would take the waters from the several ditches and carry them to an outlet into the Mississippi River at New Madrid at stages up to and equivalent to that of 1913.

To collect the waters below back-water overflow and divert them from the Lee-Rowe Ditch and James Bayou, a low-level diversion ditch was decided on. This will carry these waters to the outlet provided for St. John Bayou.

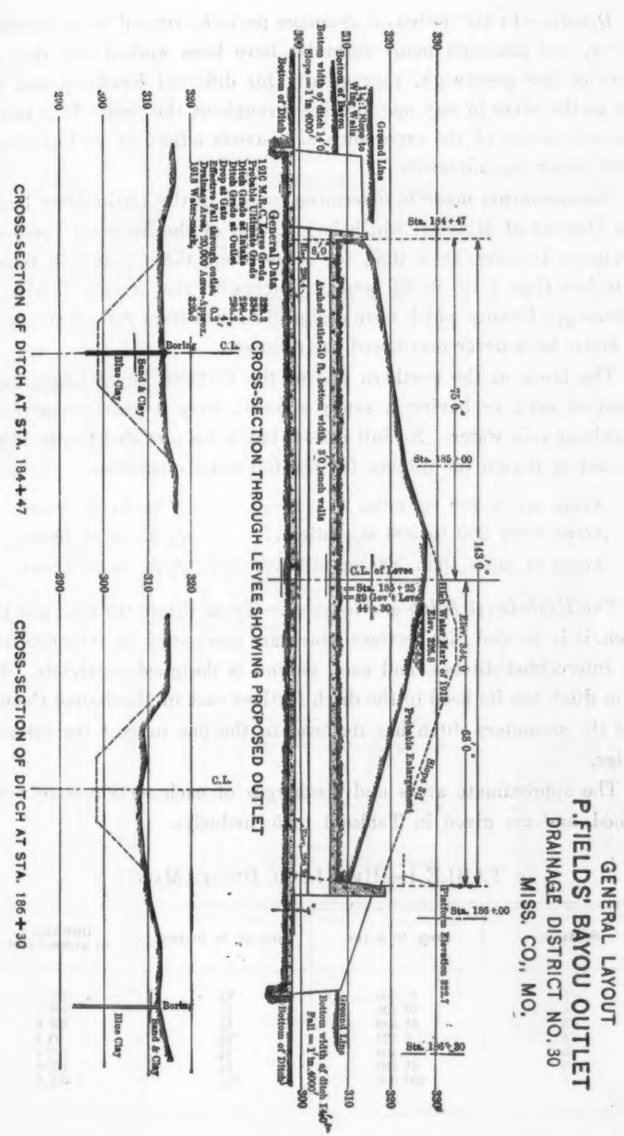

GENERAL LAYOUT
P. FIELDS' BAYOU OUTLET
DRAINAGE DISTRICT NO. 30
MISS. CO., MO.
CROSS-SECTION THROUGH LEVEE SHOWING PROPOSED OUTLET
Sta. 184+47
Sta. 185+00
Sta. 186+00
Sta. 186+30
C.L. of Levee
High Water Mark 1913
Platform Elevation 322.7
Probable Enlargement
Ground Line
Bottom of Bayou
Bottom of Ditch
Arched outlet 10 ft. bottom width, 8.0 bench walls
General Data
CROSS-SECTION OF DITCH AT STA. 184+47
CROSS-SECTION OF DITCH AT STA. 186+30
C.L.
Boring
Blue Clay
Sand & Clay

FIG. 2.

Design.—In the design of drainage projects, run-off is an uncertain factor, and although many formulas have been worked out, they are more or less guesswork, vary greatly for different localities, and will not be the same in any one locality throughout the year. It is best to take advantage of the experience of districts adjoining or included in those under consideration.

Measurements made to determine run-off in the Little River Drainage District of Missouri which is just west of the St. John Levee and Drainage District, show that, for areas where there is no hill run-off, it is less than ¼ in. in 24 hours. Ditches in the various districts of Mississippi County which were designed to care for a run-off of ¼ in. in 24 hours have never overflowed their banks.

The lands at the northern end of the District, being largely composed of sand or having a sandy sub-soil, have a great capacity for absorbing rain water. No hill run-off has to be provided for, so it was decided to design the ditches for the following quantities:

Areas up to 200 sq. miles............... ¼ in. in 24 hours.
Areas from 200 to 300 sq. miles......... $\frac{7}{32}$ in. in 24 hours.
Areas of more than 300 sq. miles........ $\frac{3}{16}$ in. in 24 hours.

The High-Level Ditch or Floodway.—To facilitate the design of this ditch, it is divided into sections which are marked by its junctions with the intercepted ditches, and each section is designed separately. The main ditch has its head in the ditch farthest east in Mississippi County, and the secondary ditch has its head in the one nearest the Sikeston Ridge.

The approximate areas and discharges of each section were ascertained, and are given in Tables 1 to 5, inclusive.

TABLE 1.—HIGH-LEVEL DITCH (MAIN).

Section.	Area, in acres.	Run-off, in inches.	Discharge, in second-feet.
1	30 000	¼	315.0
2	10 120	¼	106.2
3	31 300	¼	328.6
4	4 120	¼	43.2
5	13 400	¼	140.7
6	11 680	¼	122.6
7	250 000	$\frac{3}{16}$	1 975.0

PLATE XI.
TRANS. AM. SOC. CIV. ENGRS.
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STROHL ON
DRAINAGE PROBLEMS
IN THE MISSISSIPPI VALLEY.

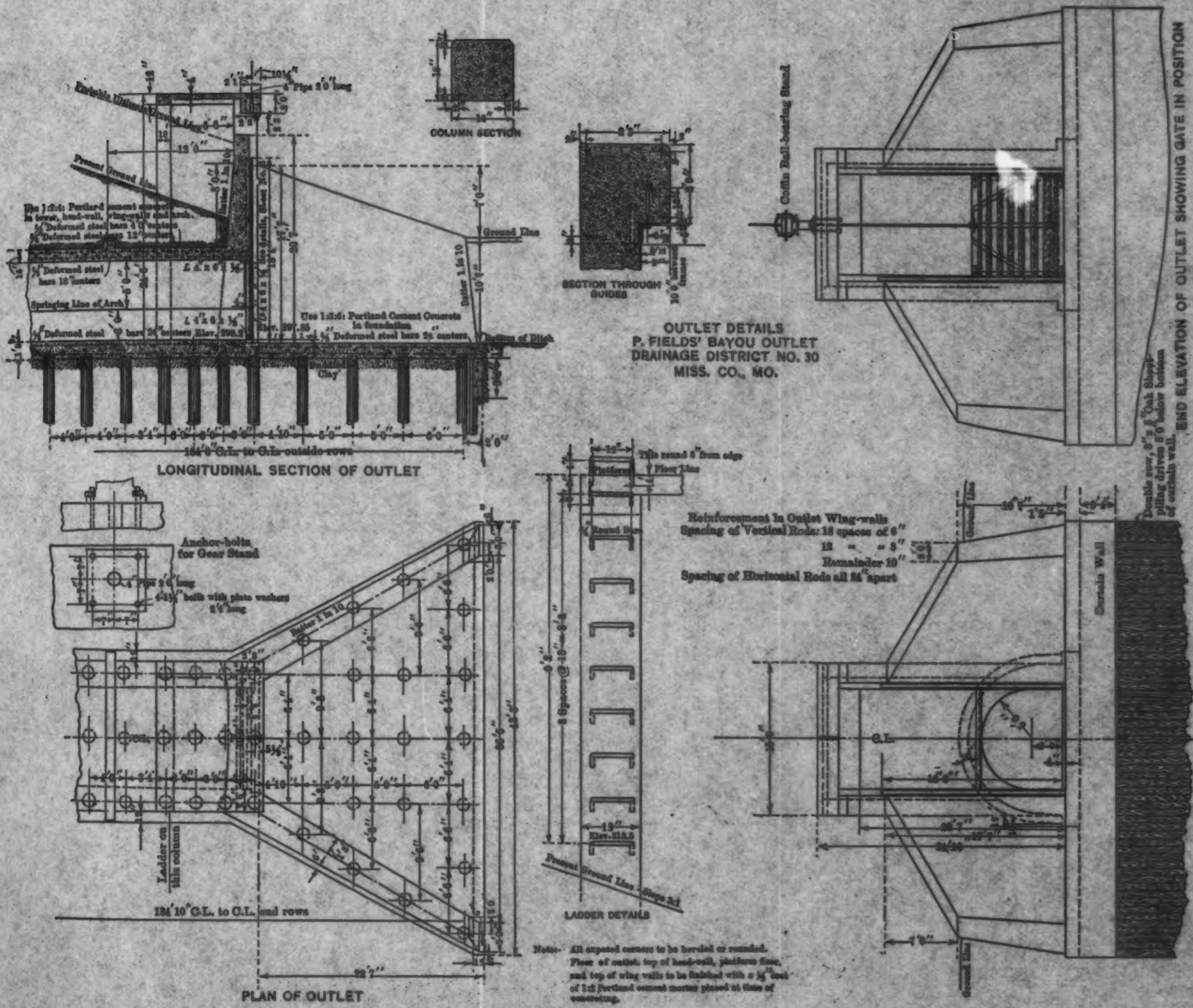


TABLE 2.—SUMMARY OF DISCHARGE FROM SECTIONS, IN SECOND-FEET (MAIN).

Section.	Discharge.	Increments.				
1	315.0					
2	421.2	106.2				
3	749.8	434.8	328.6			
4	793.0	478.0	371.8	43.2		
5	933.7	618.7	512.5	183.9	140.7	
6	1 056.5	741.3	635.1	306.5	263.3	122.6
7	1 975.0	(Main ditch below junction.)				

TABLE 3.—HIGH-LEVEL DITCH (SECONDARY).

Section.	Area, in acres.	Run-off, in inches.	Discharge, in second-feet.
1	35 340	¼	371.1
2	70 000	¼	735.0
3	12 200	¼	128.1
4	4 500	¼	47.2
5	23 000	¼	241.5

TABLE 4.—SUMMARY OF DISCHARGE FROM SECTIONS, IN SECOND-FEET (SECONDARY).

Section.	Discharge.	Increments.	
1	1 106.1		
2	1 234.2	128.1	
3	1 281.4	175.3	47.2
4	1 334.0	(145 000 acres with 7/32-in. run-off.)	

As the elevation of the grade of the bottom of the high-level ditch is fixed by the bottom of the ditch where it heads, and as it flows across the natural slope of the country, it is necessary to use very flat slopes.

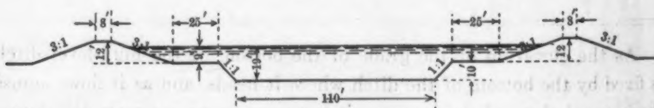
In order that the lands above back-water overflow may have an outlet at flood stages of the Mississippi River and that the lower lands will not overflow, the banks of the high-level ditch must be leveed. The grade of these levees is not less than 3 ft. above the 1913 flood-plane. In construction, it is planned to use a drag-line excavator on each bank, and build the levees with the excavated material at one operation.

Flood Flow.—To produce a flow through the ditch at flood stages, the incoming waters must rise above the flood-plane of the river until a sufficient fall is secured to produce flow through the ditch.

TABLE 5.—DESIGN OF SYSTEM.

Section No.	Estimated discharge, in second-feet.	Slope.	Bottom width, in feet.	Depth of water, in feet.	Area, in square feet.	Hydraulic radius.	Velocity, in feet per second.	Theoretical discharge, in second-feet.	Kutter's coefficient.
MAIN DITCH.									
1	315.0	0.0001	35	5.4	218.16	4.34	1.60	349.0	0.025
2	421.1	0.00008	40	6.0	276.0	4.84	1.75	483.0	0.025
3	749.8	0.00008	55	6.0	366.0	5.08	2.00	732.0	0.0225
4	793.0	0.00008	60	6.0	396.0	5.14	2.00	792.0	0.0225
5	933.7	0.00008	65	7.0	504.0	5.94	2.05	1 033.0	0.0225
6	1 056.3	0.0001	70	7.0	539.0	6.03	2.24	1 207.0	0.0225
7	1 975.0	0.0001	110	8.0	944.0	7.12	2.50	2 360.0	0.0225
SECONDARY DITCH.									
1	1 106.1	0.0001	70	7.0	539.0	6.03	2.24	1 207.0	0.0225
2	1 234.2	0.0001	75	7.0	574.0	6.05	2.24	1 286.0	0.0225
3	1 281.4	0.0001	78	7.0	595.0	6.08	2.26	1 345.0	0.0225
4	1 334.0	0.0001	80	7.5	656.0	6.48	2.37	1 555.0	0.0225

The length of the ditch is approximately 25 miles, and it is estimated that a surface fall of 0.025 ft. per mile will be sufficient to produce the required discharge. Fig. 3 shows its cross-section. Its area is 3 063 sq. ft., its perimeter, 245.16 ft., and its hydraulic radius, 12.5. Wheeler's formula, $V = 0.8 \sqrt{rs}$, gives a velocity of 0.63 ft. per sec. and a discharge of 1 930 sec-ft. The velocity by Kutter's formula is somewhat greater than 0.63 ft. per sec., therefore it is assumed that this is a safe margin.



CROSS-SECTION OF MAIN DITCH

FIG. 3.

By adding the difference in elevation of the surface between the source and the outlet, to the elevation of the 1913 flood plane at New Madrid, the approximate elevation of the location was ascertained, and the ditch was laid out along these lines, neglecting small irregularities of the ground.

Siphon Outlet for St. John Flats.—Along the western side of the District there are very low lands, known as St. John Flats. Through this territory a dredge ditch, called St. John Ditch, or Main Ditch,

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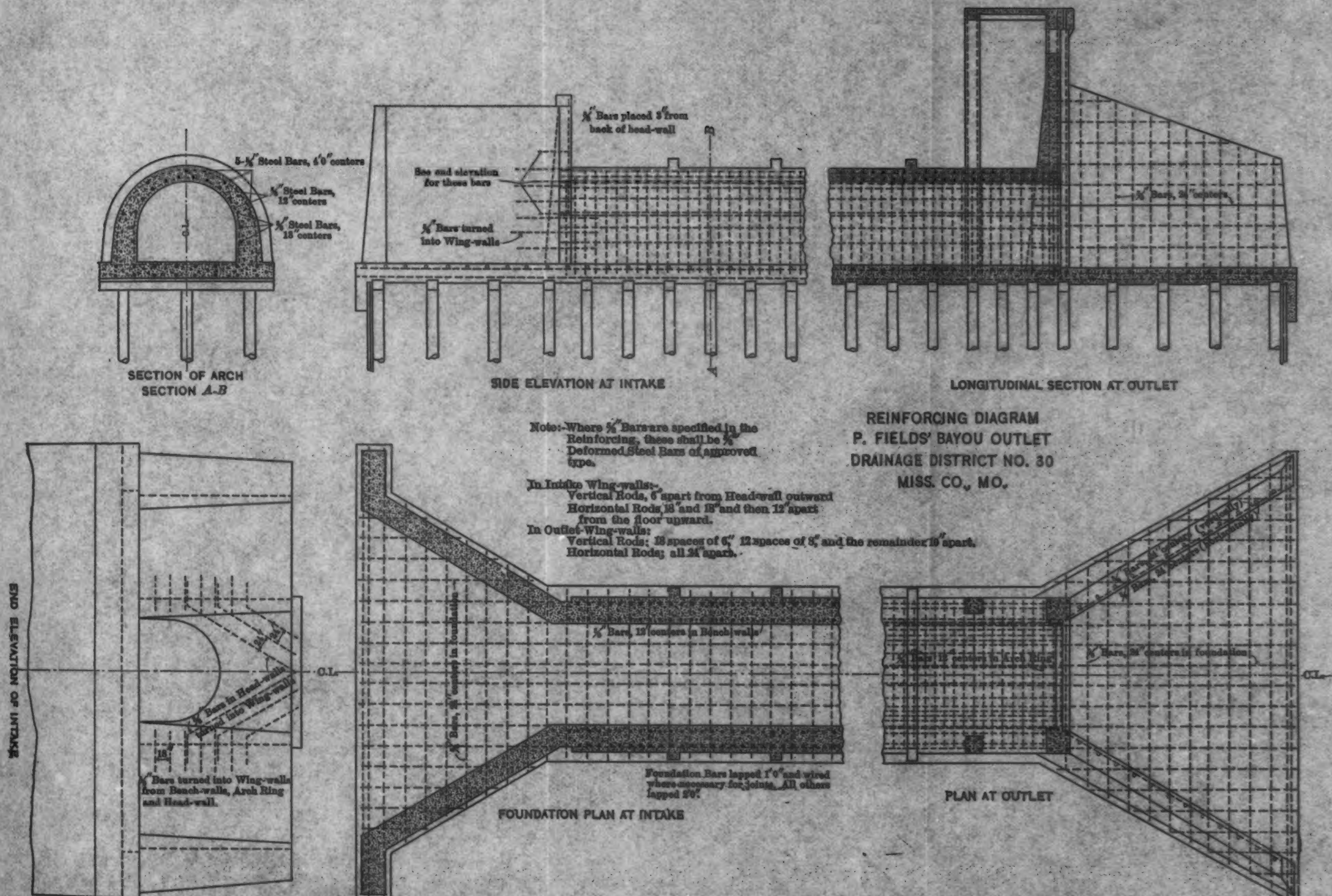

District No. 5, has been cut. It is necessary to intercept this ditch near the northwest corner of the District and to cross it with the high-level ditch near its outlet into St. John Bayou. This leaves a basin between the Sikeston Ridge and the high-level ditch without an outlet and below back-water overflow.

An inverted siphon was then planned at the crossing of the high-level ditch to transfer the waters from the basin to the outlet as before. The siphon had to be arranged so as to permit the silt that will collect therein to be easily removed, and, to this end, it was decided to use a gate to close the siphon at the upper end. A bulkhead can then be placed at the outlet of the siphon and the water and silt removed. This gate will also act as a protection to the basin in case the river-front levee should break.

Spillways.—At points where the bottom of an intercepted ditch is more than a few feet above the grade of the high-level ditch, concrete spillways or drops must be provided in order to prevent the erosion of the bottom of the ditches and the consequent silting of the high-level ditch.

At the mouth of the high-level ditch, there must be a spillway providing a drop of about 12 ft., in order to prevent the erosion of the bottom of the ditch, thus endangering the side levees.

Low-Level Diversion Ditch.—This ditch, planned to divert the waters from James Bayou and the Lee-Rowe Ditch to the St. John Bayou, also crosses the natural slope of the land, and, in locating it, natural water-courses and low lands were followed as much as possible, thus effecting an appreciable saving of necessary work.

Design.—In designing this ditch, three sections were deemed sufficient, and it was planned large enough only to take the run-off from the lands tributary to it after the construction of the high-level ditch.

TABLE 6.—LOW-LEVEL DITCH.

Section.	Area, in acres.	Run-off, in inches.	Discharge, in second-feet.
1	26 820	¼	282.0
2	43 960	¼	461.6
3	66 320	¼	096.4

Sub-Surface Flow.—It was noted, toward the end of the long dry season of 1913, that considerable water still flowed in several ditches.

During September, 1913, about the end of the dry season, discharge observations were made in the St. John Ditch at three points:

(1) At its crossing with the Iron Mountain Railway;
(2) Near its outlet in St. John Bayou; and
(3) At the mouth of St. John Bayou.

TABLE 7.—SUMMARY OF DISCHARGE FROM SECTIONS, IN SECOND-FEET.

Section.	Discharge, in second-feet.	Increment.
1	282.0	
2	743.6	461.6
3	1 100.0	137 100 acres at $\frac{7}{32}$-in. run-off.

TABLE 8.—DESIGN OF LOW-LEVEL DITCH.

Section No.	Estimated discharge, in second-feet.	Slope.	Bottom width, in feet.	Depth of water, in feet.	Area, in square feet.	Hydraulic radius.	Velocity, in feet per second.	Theoretical discharge, in second-feet.	Kutter's coefficient.
1	282.0	0.00008	20	7.5	206.25	5.01	1.59	326.0	0.025
2	743.6	0.00016	45	7.0	364.0	5.62	2.35	825.0	0.025
3	1 100.0	0.00015	70	7.0	514.0	6.09	2.47	1 270.0	0.025

The first point is about 18 miles below the head of the ditch; the second is 20 miles below the first; and the third is 7 miles below the second. The head of the ditch was encompassed, and no inflow was found; in fact, little water was discovered until a point about 10 miles below the head of the ditch was reached.

The approximate areas above each point are as follows: (1), 140 sq. miles; (2), 300 sq. miles; and (3), 330 sq. miles.

On September 3d, 1913, the following results were obtained:

at (1) 94.7 sec-ft. = 0.68 sec-ft. per sq. mile.
at (2) 111.7 " = 0.37 " " " "
at (3) 84.1 " = 0.26 " " " "

It was assumed that some error in the observation might have been made, and on September 23d, 1913, the following results were obtained:

at (1) 97.8 sec-ft. = 0.70 sec-ft. per sq. mile.
at (2) 120.8 " = 0.40 " " " "
at (3) 90.8 " = 0.28 " " " "

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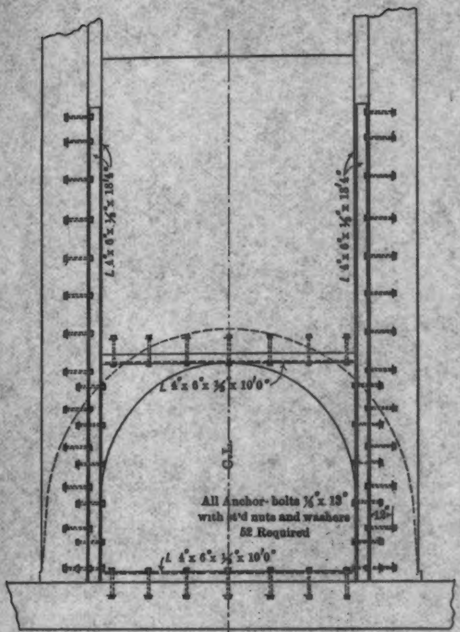
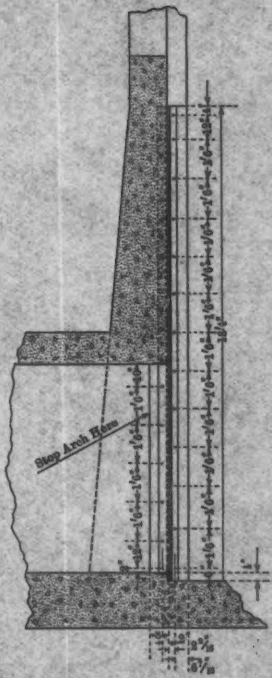
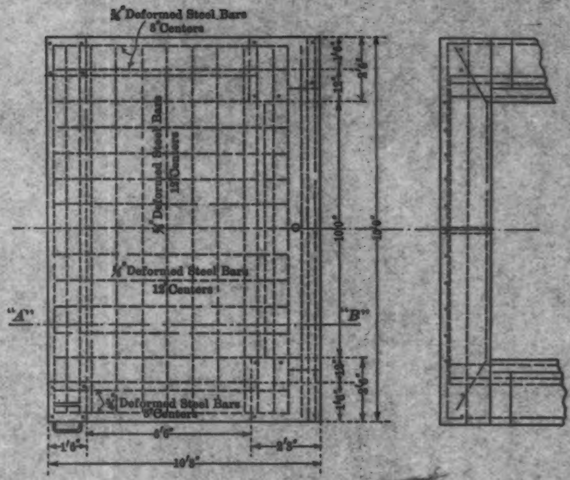


DETAIL OF GUIDE-ANGLES
AT OUTLET

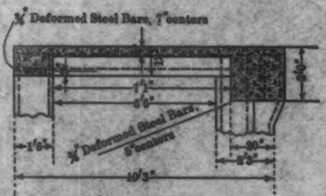


SECTION ON CENTER LINE

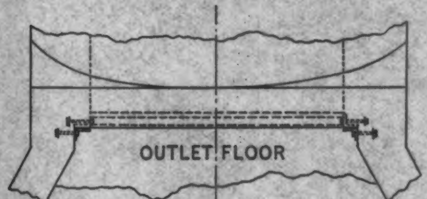
Note:-All exposed corners to be beveled or rounded
Platform floor to be finished with a ½" coat
of 1:2 cement mortar placed at time of
concreting.



PLATFORM REINFORCING



SECTION *A-B*



OUTLET FLOOR

DETAILS
P.FIELDS' BAYOU OUTLET
DRAINAGE DISTRICT NO.30
MISS. CO., MO.

Table 9 gives the monthly rainfall records at New Madrid and at Cairo, stations at opposite corners of the District, from April, 1913, to September, 1913, inclusive.

TABLE 9.—RAINFALL AT NEW MADRID AND CAIRO.

Station.	April.		May.		June.		July.		August.		Sept.	
	Rainfall.	Average.	Rainfall.	Average.	Rainfall.	Average.	Rainfall.	Average.	Rainfall.	Average.	Rainfall.	Average.
New Madrid....	3.96	3.88	1.58	4.90	1.27	4.40	2.73	4.38	1.09	3.79	5.41	2.95
Cairo..........	2.27	3.57	0.87	3.83	1.07	4.31	4.19	3.45	1.54	2.93	2.22	2.47

It is generally believed that the sand stratum beneath the high lands becomes saturated with water when the river is at high stages, and the water gradually drains out during the summer. The drop between the results obtained at Points 2 and 3 has not yet been explained satisfactorily.

Drainage of Ridge Lands and New Madrid.—The construction of the high-level ditch along the Sikeston Ridge and the river levee at New Madrid obstructs the drainage, and a system of collecting ditches and inverted siphons has been planned for the relief of these points.

Pumping Plants.—It is physically possible to carry the water collected by the low-level ditch through the Sikeston Ridge to an outlet through the St. Francis River and thus eliminate all pumping costs, but, under the present legal conditions, this outlet cannot be effected.

It is necessary, therefore, to provide adequate pumping facilities before the reclamation of the low lands is complete; owing to the undeveloped state of these lands, however, it is not advisable to put the cost thereof on such lands at present. The area of this pumping territory is approximately 140 000 acres. The run-off of approximately 46 000 acres can be accommodated in storage.

Storage Basins.—The Ten Mile Pond Basin includes what is known as Ten Mile Pond, Four Mile Pond, Six Mile Pond, Eagle Pond, and various smaller ponds and lakes. These will operate as one basin because James Bayou will act as a connecting or equalizing channel throughout this territory.

A storage level at Elevation 299.0 has been decided on, because that will give the greatest storage with the least damage to adjoining lands. The total area covered by this storage basin will approximate 6 820 acres, all of which, except 1 300 acres, will have no water except immediately after hard rains and at times when the flood-gates in St. John Bayou have been closed for a long period.

The storage in this basin will amount to approximately 12 250 acre-ft. The discharge capacity of the low-level ditch is somewhat greater than 1 500 acre-ft. per day; therefore, the impounded water would be discharged in approximately 6 days. To accomplish this, two spillway dams with flood-gates will be required. At ordinary times these gates will be left open to pass the natural drainage. When the gates in St. John Bayou are closed, the spillway dam gates will be closed. If the water rises above the crest of the spillway, which will be placed at an elevation of 299.0 ft., it will overflow and pass on down the low-level ditch and be held behind the gates in St. John Bayou.

One dam is to be placed in the Lee-Rowe Ditch near the point where it leaves Ten Mile Pond, and the other near the head of the low-level ditch. No land between the two dams is at an elevation lower than about 303.0 ft.; the land south of the dam in the low-level ditch is still higher, and no water can pass from the basin to a lower level except over the spillways.

Eagle Nest Cypress Basin is near the center of Township 23 North, Range 15 East, and is bounded on the north and west by a high bank, averaging about 14 ft. above the bottom of the basin. The ground rises gently from the bottom of the basin in a southeasterly direction. The bottom of this basin is at an elevation of about 286.0 ft., and the ground at the Mississippi River is at an elevation of 300.0 ft.

This basin is separated from the flat lands along the St. John Ditch and St. John Bayou by a ridge varying in elevation from 296.0 to 298.0 ft. The East Bayou cuts through this ridge, connecting it with St. John Bayou.

The flats along St. John Ditch are at an elevation of from 292.0 to 295.0 ft., therefore, without some kind of a dam in East Bayou, a storage level at an elevation of 292.0 ft. could not be reached without encroaching on the lands along St. John Ditch and St. John Bayou.

With a dam across East Bayou, a storage level of 296.0 ft. could

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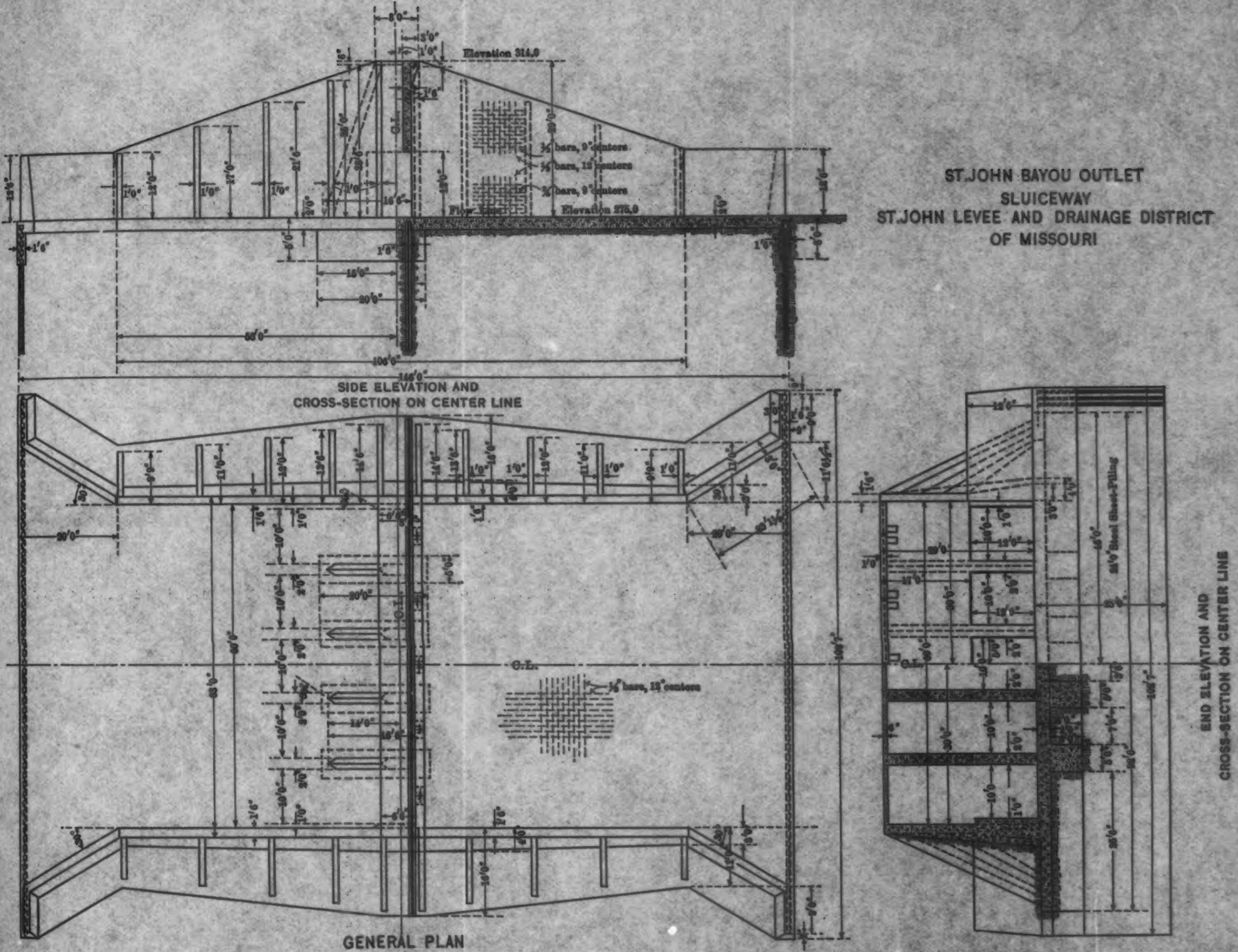
ST. JOHN BAYOU OUTLET
SLUICEWAY
ST. JOHN LEVEE AND DRAINAGE DISTRICT OF MISSOURI



be maintained, but practically all the lands from the ridge to the river would be overflowed.

If a storage level is placed at an elevation higher than 292.0 ft., it is doubtful whether the cost of flood-gates and dam and reservoir would be warranted by the storage obtainable, hence no plan is proposed to use this basin for storage.

Storage Observations.—Observations to determine the quantity of rainfall that will reach the storage basins have been made for the Big Lake Drainage and Levee District of Missouri, which adjoins the St. John Levee and Drainage District of Missouri on the north, by L. T. Berthe, Assoc. M. Am. Soc. C. E., Engineer for that district. The results of these observations show that only about 21% of the rainfall finally reaches the basin.

The records show that the flood-gates of St. John Bayou would be closed on an average about 90 days each year, also that the average rainfall for this period has been 0.123 in. per day. Assuming that 25% of the rainfall will reach the basins, there will be 10 598 acre-ft. in storage, and under these conditions the run-off from 46 000 acres will be cared for.

These conditions, of course, are taken as an average, but a variation either way may be expected.

Flood Frequency.—All work for the district has been based on the flood heights as recorded for the New Madrid gauge. Table 10 shows the number of days each year that the river was above the different stages, and hydrographs for the New Madrid and Cairo gauges have been plotted.

An interesting comparison of the gauges is shown by the hydrographs, and the variation of the readings of the two can be compared for different stages of the river. Zero of the New Madrid gauge is at an elevation of 262.59 ft., Memphis datum, and that of the Cairo gauge at an elevation of 277.71 ft.

Table 11 shows the number of years that each stage has been reached out of the total number of years that the records have been kept, and gives the best possible conclusion as to the frequency of the various stages.

Estimates.—Yardage estimates for the levee construction show 6 197 500 cu. yd. of embankment. This work is variously estimated at

from 12 to 25 cents per cu. yd., depending on the class of work and the quantity of grubbing.

TABLE 10.—RIVER STAGES AT NEW MADRID.

Year.	Number of Days Above River Stage of:							
	32 ft.	34 ft.	36 ft.	38 ft.	40 ft.	42 ft.	44 ft.	46 ft.
1880			No records at New Madrid.					
1881			"	" "	"	"		
1882			"	" "	"	"		
1883			"	" "	"	"		
1884	56	46	36	16	11	0	..	..
1885	0	..	..	..	..	..	..	..
1886	27	24	20	15	5	0	..	..
1887	43	38	24	0	..	..	..	..
1888	22	12	1	0	..	..	..	..
1889	0	..	..	..	..	..	..	..
1890	64	52	37	3	0	..	..	..
1891	67	53	8	0	..	..	..	..
1892			No record at New Madrid.					
1893	61	33	16	6	0	..	..	..
1894	0	..	..	..	..	..	..	..
1895	0	..	..	..	..	..	..	..
1896	0	..	..	..	..	..	..	..
1897	59	54	46	33	8	0	..	..
1898	38	28	15	6	0	..	..	..
1899	43	34	25	0	..	..	..	..
1900	1	0	..	..	..	..	..	..
1901	13	4	0	..	..	..	..	..
1902	8	0	..	..	..	..	..	..
1903	78	61	18	12	0	..	..	..
1904	30	20	14	5	0	..	..	..
1905	20	13	0	..	..	..	..	..
1906	22	18	10	0	..	..	..	..
1907	51	32	16	10	0	..	..	..
1908	83	50	16	0	..	..	..	..
1909	66	40	23	12	0	..	..	..
1910	14	4	0	..	..	..	..	..
1911	16	12	5	0	..	..	..	..
1912	71	57	47	36	23	16	2	0
1913	62	55	49	39	20	15	8	0
1914	9	0	..	..	..	..	..	..

TABLE 11.—FLOOD FREQUENCY AT NEW MADRID.

Gauge reading.	Number of years stage was reached in 29 years.	Corresponding elevation.
32	25	294.59
34	22	296.59
36	19	298.59
38	12	300.59
40	5	302.59
42	2	304.59
44*	2	306.59

*The highest recorded reading on the New Madrid gauge is 44.6 ft., corresponding to an elevation of 307.19. This stage was reached on April 9th. 1913.

The total excavation for the various ditches is placed at 10 715 000 cu. yd., and this work is estimated at from 8½ to 18 cents per cu. yd. Clearing of right of way is estimated separately.

The total cost of the improvement, as planned, without the pumping plant, but including all dams, flood-gates, sluice-ways, siphons, etc., is $3 750 000.

Order of Construction.—On account of the cost of the system, it is certain that the construction of the high-level ditch will be delayed until some future time. The work decided on at present consists of the extension of the levee about 17 miles down the river from Dorena, the cutting of the low-level ditch, and the construction of a levee along the river front at New Madrid and along the east edge of the Sikeston Ridge to high land above overflow, in order to prevent the passage of water across the Sikeston Ridge at high stages of the river.

The work outlined will fit in with the complete plan, with the exception of that part of the Sikeston Ridge Levee which extends along the ridge beyond the location of the high-level ditch.

It is necessary to investigate the probable effect of this procedure, as no increase in the size of the low-level ditch is contemplated, although its contributing area is greatly increased.

Approximately, 172 600 acres will now be tributary to the Ten Mile Pond Basin, and, at $\frac{7}{32}$ in. run-off in 24 hours, the inflow to the basin will be 1 364 sec-ft.

The estimated discharge of the low-level ditch, with water running 7 ft. deep, is 825 sec-ft., but where the ditch leaves the Lee-Rowe Ditch a depth of 14 ft. can be reached without flooding the banks.

The water will back up in the Lee-Rowe Ditch and the Ten Mile Pond Basin until it attains a depth in the low-level ditch sufficient to produce a discharge equivalent to the volume of water entering the basin, after which no additional rise will occur. A depth of 9.5 ft. in the ditch will produce this discharge, and will produce a water level at an elevation of about 296.0 ft. in the ditch; the level in the basin will then probably reach 298.0 ft.

It is believed that, with the low-level ditch alone in operation, under the usual conditions, little or no damage will result, except in cases of excessive rainfall.

A maximum rainfall of 9.6 in. in 48 hours has been recorded for these localities. Assuming that 25% run-off results, this will produce 0.2 ft. in 48 hours, or 8 700 sec-ft. With a depth of flow of 13.5 ft. in the low-level ditch, a discharge of 2 730 sec-ft. results, and the excess,

of about 6 000 sec-ft., must be held in storage until the low-level ditch has again emptied the basin, which will take about 7 days.

A depth of flow of 13.5 ft. in the low-level ditch is attained with a water level of about 300 ft., and some flooding of the lands above 300 ft. will occur.

The low-level ditch in Eagle Nest Cypress Basin has an effective depth of only 5 or 6 ft., and the ditch discharge here is 655 sec-ft. With the usual conditions of run-off ($\frac{7}{32}$ in. in 24 hours), the ditch brings to the basin 1 364 sec-ft., so it is seen that some flooding of the lands along the banks will occur, but, under the condition that the ditch brings 2 730 sec-ft. to the basin, a ponding will occur until the ditch leading from the basin attains an equivalent discharge, which will be reached with a water level at 293.0 ft. elevation and a depth of 14 ft. in the ditch. Therefore, under the maximum conditions, all lands in this basin below an elevation of 293.0 ft. will be flooded.

The frequency of these excessive storms must also be considered. The records for Missouri show that a maximum hourly rainfall of 4.74 in. has been observed. The maximum 1-day storm recorded shows a rainfall of 8.0 in., and 16 storms with a rainfall of 5 in. or more in 24 hours have been observed. The maximum 2-day storm recorded shows a rainfall of 9.6 in., and 17 storms with a rainfall of 5 in. or more in 48 hours have been recorded.

The records of the Cairo Station, kept since 1872, show thirty-five 1-day storms with a rainfall of 2 in. or more; fourteen storms with 3 in. or more; six storms with 4 in. or more; and two storms with 5 in. or more.

Assessments of Benefits.—The Missouri laws provide that the cost of construction must be apportioned over the lands or other property receiving benefits through the construction of a plan for reclamation, in the proportion of the benefits received. A tax equal to the amount of benefits can be collected, but where the benefits received only equal the cost of construction, it is a useless expenditure of money, and the Court will not recognize the project.

Three commissioners are appointed by the Court, under which the District is organized, on the filing of the plan for reclamation. It is their duty to go on the ground to determine the value of all property or lands to be acquired for right of way or other works of the District, as set out in the plan for reclamation, either within or without the Dis-

trict. They shall assess the amount of benefits and damages, if any, that will accrue to lands or other property from carrying out and putting into effect the plan for reclamation.

The chief engineer of the District or one of his assistants shall accompany the Commissioners at all times when engaged in their duties, and shall render his opinion in writing when called for.

The following outline, submitted by the writer, for the assessment of benefits in this District was adopted by the Commissioners, and will show the part taken by the engineering features of a plan for reclamation in the assessment of benefits:

The plan adopted for this District does not provide complete reclamation for all the lands within the District, but the proposed work when completed will form a part of a general plan providing complete reclamation. The construction of the Sikeston Ridge Levee will protect the lands on the ridge and in the City of New Madrid from overflow, and the benefits derived by the ridge lands will be those resulting from protecting them from the scour to which they are subjected by the severe current passing over them at high-river stages. The damage that this current has caused, by washing away the rich soil, cutting deep holes and channels across the lands, washing out roads and railroads, tearing down fences, and moving buildings from their foundations, etc., property damage in the City of New Madrid, and loss of merchandise, can be estimated at nearly \$300 000 for 1913 alone.

The ridge lands and the city can be considered separately from the low lands, as the proposed work is not connected.

Benefits to New Madrid.—The different parts of the town are practically at the same elevation, and will be overflowed at the same stage of the river, so that all property may be considered equal in this respect. The damage done by the water, then, will depend on the value of the property. No personal property can be assessed.

It will be necessary to make a complete valuation of all real property within the city limits, placing a fair value on each lot or tract and adding thereto the value of buildings thereon. A representative property can then be selected and the flood damage to it estimated. This damage then will be the total which the property will sustain during a period determined from past overflows, which is about 7 years. The damage divided by 7 and this amount capitalized at 6% will be the amount of the benefit. A percentage is then found between

the benefit and the value of the property, and this percentage of the value of any property is taken as the benefit to that property.

Ridge Lands.—The benefits to ridge lands are determined in part by the elevation of the respective tracts. Lands on the east side of the ridge being somewhat lower, are subject to overflow before those on the summit and west side. The lands at a higher elevation are subject to the severe cutting of the current, as are those on the west side of the ridge. The benefit to a tract of land, therefore, will depend largely on the elevation and location and the probability of overflow, and should vary from $5 to $30 per acre.

Bottom Lands.—The overflow lands of the District are for the greater part not under cultivation, but are timber or cut-over lands. The value of those which are under cultivation will range from $35 to $45 per acre, and that of the remainder varies from $5 to $15, depending on the quantity of timber thereon.

As the plan for reclamation for this District proposes to extend the levee down the river from Dorena about 17 miles, there is still a gap, between the end of the levee and the levee at New Madrid, of about 6 miles. This leaves some of the lands of the District still subject to overflow from back-water from the Mississippi River.

It is not possible to show a benefit to the lands above back-water overflow, therefore, although there is a large area in the district above overflow, it cannot be assessed for benefits under this plan.

As the lands assessed are subject to overflow from back-water, it will be necessary to consider the effect of the extension of the levee on the water level over these lands at flood stages of the river.

During recent overflows, no true back-water elevations were obtained, as there were several breaks in the levees above Dorena, and the water from the Mississippi above Dorena raised the level of the water over these lands.

What the result of the 1913 flood would have been had the levees held is a matter of conjecture. The water level at Dorena was established at approximately 315.6 ft., but, had there been no breaks in the levees above, it would have undoubtedly reached an elevation of 316.0 ft., and this will be taken as the possible elevation of the back-water on the lands of the District.

Just what difference would be found between this level and the level of the water at the extremity of the overflow is not known, but it is

certain that the elevation at the limit of overflow will not be quite as high as that at its origin. From the information at hand, it may be safely stated that this difference will be about 1 ft. in 9 miles, depending, of course, on the duration of the flood. The shorter the flood period, the greater the difference will be found, and if the flood is of sufficient duration, the water may seek nearly a level surface.

With the extension of the levee as proposed, the origin of the back-water will be 17 miles farther down stream. Considering the fall of the river and the increased flood height with the waters confined, the elevation of the back-water at the origin will be reduced from 316.0 ft. to approximately 310.0 ft., and the same differences will result on the level of the back-water as before.

The 1913 flood is considered to have been one of the maximum floods in the Mississippi Valley. Floods which average nearly as high as that of 1913 are by no means rare, as those of 1882, 1883, 1884, 1897, 1903, 1907, and 1912 were far above the average years.

The construction of the levees at Cairo and along the Mississippi River has also increased the flood height of the more recent overflows, so, under the same conditions, the earlier floods might have reached the same stage as that of 1913.

The frequency of floods must be considered, as lands subject to overflow every year would receive far greater benefit than those subject to overflow but once in 10 years, were the overflows taken away completely.

Therefore, it must be seen that the benefits which a tract of land receives from this improvement depends: (1), on the elevation of the tract; (2), on the frequency of overflow; and (3), on the improvements on the tract.

By comparing the stages of the river at New Madrid with the elevation of the flood-plane at the end of the proposed levee, and taking the frequency of these stages, the probable stage of overflow of lands at various elevations may be determined and the period between overflows taken from the flood frequency.

By comparing the elevation of the back-water from Dorena with that from the end of the proposed levee, and combining with the frequency, we have the following scale of assessment:

Lands with elevation of from 294.0 to 296.0, 5% of maximum benefit.

Lands with elevation of from 296.0 to 297.0, 10% of maximum benefit.

Lands with elevation of from 297.0 to 298.0, 20% of maximum benefit.

Lands with elevation of from 298.0 to 299.6, 30% of maximum benefit.

Lands with elevation of from 299.6 to 300.6, 40% of maximum benefit.

Lands with elevation of from 300.6 to 302.6, 50% of maximum benefit.

Lands with elevation of from 302.6 to 304.6, 75% of maximum benefit.

Lands with elevation of from 304.6 to (306.6-309.0), 90% of maximum benefit.

Lands with elevation of from (306.6-309.0) to 310.0, 50% of maximum benefit.

Lands with elevation of from 310.0 to 312.0, 30% of maximum benefit.

Lands with elevation of from 312.0 to 314.0, 10% of maximum benefit.

Lands with elevation of from 314.0 to 315.0, 5% of maximum benefit.

TABLE 12.—ELEVATION—FLOOD-FREQUENCY TABLE.

New Madrid gauge reading.	Number of years that this stage was reached in 29 years past.	Corresponding elevation at New Madrid.	Corresponding elevation at end of proposed levee.	Corresponding elevation at Dorena, inside of levee.	Corresponding elevation at Grassy Pond Cypress.	Corresponding elevation at Selkirk.	Present water level at Dorena.
32	25	294.59	296.5	295.0	294.5	295.5	299.6
34	22	296.59	298.5	297.0	296.5	297.5	302.1
36	19	298.59	300.6	299.1	298.6	299.6	304.6
38	12	300.59	303.0	301.5	301.0	302.0	307.1
40	5	302.59	305.8	304.3	303.8	304.8	309.6
42	2	304.59	308.0	306.5	306.0	307.0	312.4
44	2	306.59	310.0	308.5	308.0	309.0	315.1

An interesting point in this scale is the peak in the benefits. The value per acre of absolute protection from overflow was taken at $24, and the different rates derived.

Lands in Levee District No. 1, Mississippi County, having borne the cost of construction of the levees along the Mississippi River, which are now to become a part of the system and lessen the cost of protection to lands outside of Levee District No. 1, must be allowed credit in reduced benefits for the work constructed by them. This credit is to be worked out as follows: Determine the cost of these levees and the benefit derived by Levee District No. 1 lands. Determine the proportion of the benefits of the proposed improvements between the lands in Levee District No. 1 and all others; then this proportional part of the benefits due to the existing levee will give the reduction due to lands in Levee District No. 1. As the plan includes the cutting of the low-level ditch, an equitable adjustment of its cost must be made.

It would appear that, before the opening at James Bayou and the Lee-Rowe Ditch can be closed and any benefits derived from the construction of the levee, this ditch is necessary. Little benefit is derived from the use of this ditch for local drainage by any lands, and such benefits should be distributed over the lands getting levee benefits and in the same proportion as the latter. This ditch cannot be considered as a drainage project, as it is made necessary absolutely by the construction of the levees and, therefore, is properly chargeable as a levee project.

A part of the Lee-Rowe Ditch is used under this plan for the low-level ditch, and the cost of the section used was about $8 000. It may be considered that, as the cost of closing this ditch was in excess of the cost of the section, and as the cutting of this ditch directly into the river was a damage to a large part of the lands in the district, the lands responsible for the cutting of the Lee-Rowe Ditch are not entitled to any credit for the part used.

Railroads and Other Property.—The Missouri law provides that railroads shall be assessed according to the increased physical efficiency and decreased maintenance cost by reason of the protection to be derived from the proposed works and improvements.

Two railroads, the St. Louis and San Francisco and the St. Louis Southwestern, suffered considerable loss due to flood damage and loss of traffic during the past two high waters. Several washouts occurred which they found necessary to span by pile trestles in order to get their tracks open in the least possible time after the flood passed.

These trestles are no longer necessary, and the benefit derived from closing them was worked out in a tabular form, as follows: Column 1, showing the length of trestle filled; Column 2, the yardage in the fill; Column 3, the cost of embankment; Column 4, maintenance cost of trestle over embankment at $1 per ft. per year; Column 5, maintenance cost capitalized at 5%; Column 6, net benefit by eliminating trestle, the difference of cost in Columns 3 and 5.

The benefits derived by the railroads through the elimination of flood damage and loss of traffic can be estimated only roughly.

No records of the cost of track renewals and repairs are available to the District. These records should be produced in Court in order that a correct basis may be established for the assessment. The loss of traffic cannot be fixed definitely, as can the flood damage, but it can be estimated by those who know the conditions of train operation, such as freight and passengers handled.

The total of flood damage and loss of traffic, distributed over the period of years that this damage may be expected to occur, and capitalized at 5%, will be the benefit assessed against the railroad roadway and other property.

Telegraph and telephone lines operating within the overflow are subject to damage to toll lines, by the breaking of the lines and loss of tolls, due to the population leaving the belt of danger, and some assessment of benefits should be placed on those properties.

The law provides for the assessment of public roads and, in some cases, this is done. In this case there is little benefit to be derived, as most of the roads are still subject to overflow and are not improved, so that it is not advisable to assess them.

The plan of the high-level ditch and the organization of the three counties into one district for increased efficiency in maintenance of protection works was originated by L. T. Berthe, Assoc. M. Am. Soc. C. E., who has aided the work for this district as far as possible.

J. A. Ockerson, Past-President, Am. Soc. C. E., Member of the Mississippi River Commission, was retained as Consulting Engineer.

The writer has had charge of this work for Charles H. Miller, M. Am. Soc. C. E., of the Miller Engineering Company, of Little Rock, Ark., to whom he is indebted for assistance and advice.

DISCUSSION

VERNON M. EAGER,* ASSOC. M. AM. SOC. C. E. (by letter).—This paper is, indeed, one of interest and value to those engaged in such operations, and is welcomed in a field in which the demand for practical information so far exceeds the supply, as it does in this class of reclamation. Mr. Strohl is to be highly complimented on the able manner in which he has presented the subject. Mr. Eager.

In the development of the details of the P. Field's Bayou outlet, would not an automatic gate or pair of gates prove more efficient for the purpose of maintaining the low level of the interior water than the type adopted? The paper is not quite clear on this point, and it appears to the writer that the period of play of an automatic gate under the conditions here imposed would be longer, and in many instances more effective, than that of a hand-operated gate.

The writer notes with interest the apparent diminution of subsurface flow at Point 3 on the low-level ditch. Would this not indicate the existence of a sub-stratum of sand or gravel or a sub-surface channel extending to, and sloping toward, the bed of the Mississippi River? If such a condition exists, the discharge capacity of such channel or channels would undoubtedly tend to increase with the flow, until such time as their effectiveness is impaired or destroyed by human agencies.

J. S. SPIKER,† M. AM. SOC. C. E. (by letter).—Much credit is due to Mr. Strohl, from engineers engaged in drainage and levee work, for the information contained in this paper. It is especially interesting to the writer because he has known the district, in a general way, since 1892, that is, prior to any drainage work in Southeast Missouri. His detailed knowledge of the work proposed does not justify any serious criticism of the plans as outlined in the paper, for, as a whole, the latter is worthy of commendation. The general information contained therein is systematically arranged, and the reason for an economical design is ably presented. Mr. Spiker.

The paper states that the drainage district is divided into two parts, and that the assessments of benefits in the districts are based on different conditions. In one district the benefits are reckoned on the damages done by high waters passing over and washing the high land, the lots, and improvements in the City of New Madrid, and, in the other, on the prevention of overflow from bank water.

The assessments of benefits in the City of New Madrid are made tangible and definite by the method presented, but the justice and

* Cathlamet, Wash.

† Vincennes, Ind.

Mr. Spiker. equality of the assessments, by such method, would not be substantiated by many of the Courts.

Some judges have held that the improvements on a piece of real estate and the purpose for which such improvements are used, bear the same relation to the real estate that personal property bears to the private individual; and the benefits derived by a method based on the present value of real estate and improvements, in drainage and levee proceedings, are open to criticism and revision by the Courts.

The substantial and impartial method of ascertaining the amount of benefits or damages to any piece of property or real estate, through and by any improvement, should be based on a reasonable and fair value before and after the improvement is made. If the value before is greater than the value after the improvement, the difference should be awarded as damages to the property, but if the value after is greater than the value before, the difference should be awarded as benefits.

In placing values on property or real estate before the improvement is made, one should be governed, commercially, by what the property can be used for, and not by what it is used for, in its present condition; and the same method should be used in placing values after the improvement is made.

The writer has no fear of contradiction, from those who have had experience in making assessments for drainage improvements, when he asserts that, in more than 75% of the individual pieces of property or real estate examined, the benefits are in reverse proportion to the value before the improvement; that is to say, the less the value before the improvement, the greater the proportionate benefits from such improvement.

The method of making the assessment of benefits on what is called "ridge land" is variable, resulting in values ranging from $5 to $30 per acre, depending largely on the elevation, location, and the frequency of overflow, which are the three essential elements to be considered. The variation of from $5 to $30 per acre seems excessive for land receiving complete protection from overflow.

It is presumed that, in a general way, the lower the land the higher the assessment per acre, was the method used in making the assessments. This, the writer regrets, was his first experience. Twenty-six years of practical experience and observation of leveed districts have caused him to change radically his method of making assessments. The agricultural and commercial advantages of the high land, over the low land, are so perceptible, after the improvement is made, that additional reasons are superfluous. The bottom land assessments are based on the benefits derived from the prevention, or partial prevention, of overflows from back-water.

It is presumed that in the St. John Levee District all head-waters have been cut out by the construction of former levees, and that all the lands in this District have contributed their proportional part toward their construction. This District is to pay for the extension of the present levee toward New Madrid for a distance of 17 miles, and for the construction of the low-level diversion ditch. Justice and equity (the writer might say, the drainage and levee law) should demand, of all the drainage districts emptying water into this District, an equitable proportion of the expenses for the construction of the diversion ditch, or that they should pay damages for overflowing the lower land by their surface water; but, under the present system, the high land can say to the low land, and to the Courts: Mr. Spiker.

"Some time in the future, we are going to construct a high-level diversion ditch to take care of our water, and until we get ready to do this, we will use the low land as a dumping ground for our water, so that when we do get ready, you can help us pay for our ditch, as we can then prove that you will be benefited."

"Human laws, though designed to secure justice, are of necessity, imperfect, and hence what is strictly legal is at times far from being equitable and just." (Webster.)

The schedule of percentages is based on reasonable assumptions, namely, the elevation of the water at the origin and the duration of its extreme height.

The relative difference in elevation of the water at the origin and the extreme point of overflow will not be as great on the Mississippi as on smaller streams, owing to the duration of the extreme height of the flood-waters. On the Wabash River below Vincennes, Ind., the difference is about 1 ft. in 5 miles.

Both gravity and hydraulic reasons cause a marked difference between the Mississippi and Wabash Rivers, relative to back-water curves, on account of the fall and the duration of floods.

The schedule of percentages in connection with Table 12 and the peak of assessments is very interesting. The information is concise, comprehensive and tangible, and the writer is of the opinion that some changes might be made in the percentage, from the peak to the high land; but, on close investigation, this would only be a difference of opinion which might resolve itself into further complications.

All drainage and levee laws permit assessment of benefits to public highways and railroads, and, in some cases, to telephone and telegraph companies, but the latter is not so apparent, for railroads and highways, in many State laws, are named as essential factors of public utility, and if the proposed improvement does not benefit public highways, it causes the loss of one of the essential elements for the improvement.

Mr. Spiker. It is not the writer's intention to criticize the matter presented, but rather to commend it, and extend to all engineers who are engaged in drainage work his congratulations, for they are doing work which is a co-ordinate of Creation, when properly correlated. It is hoped that in the future many more valuable papers on drainage may be presented before this Society.

Mr. Strohl. R. M. STROHL,* ASSOC. M. AM. SOC. C. E. (by letter).—It has been deemed better practice along the Mississippi River to use flood-gates operated by hand or power than to use automatic gates. This is due to the desire to secure positive action of the gate where large quantities of drift are floating in the canals and the river (as is the case in a newly cleared territory), and also to allow the gate tender to remove any drift which may lodge in the outlet while closing the gates, rather than to trust that it will pass through and not interfere with their automatic action.

It was unfortunate that the original outline of limits for the district was not followed, but the district attorneys were of the opinion that it would cause delay in Court to have the district so formed, and advised the change which was adopted by the supervisors.

The assessment features were adjusted to meet the conditions, as far as possible, but, as is always the case where there are numerous districts with their local assessments, each attempting to form a link in a chain of levees, some discrepancies will result.

The results found in the back-water curve correspond well with that found by Mr. Spiker, making allowance for the different lengths of flood periods.

* Genoa, Ohio.

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TRANSACTIONS

Paper No. 1341

CINDER CONCRETE FLOOR CONSTRUCTION BETWEEN STEEL BEAMS*

BY HAROLD PERRINE AND GEORGE E. STREHAN, JUNIORS, AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. K. M. BOORMAN, T. HUGH BOORMAN, MYRON S. FALK, GUY B. WAITE, ALBERT OLIVER, EMILE REED LOW, ARTHUR H. DIAMANT, IRA H. WOOLSON, J. R. WORCESTER, GARDNER S. WILLIAMS, A. L. A. HIMMELWRIGHT, JOHN B. HUTCHINGS, JR., AND HAROLD PERRINE AND GEORGE E. STREHAN.

SYNOPSIS.

This paper treats of the special form of cinder concrete fire-proof floor construction between steel beams which is used extensively in New York City and elsewhere. It comprises a brief general discussion of the characteristics of the material, with reference to fire resistance, corrosion, strength, etc. The major portion, however, deals with the results of an extensive series of tests on cinder concrete, plain and reinforced, both in the form of cylinders and typical slabs, as a means of obtaining sufficient accurate basic data for use in compiling possible future regulations. The tests were conducted by the Department of Civil Engineering, Columbia University, partly in co-operation with the Bureau of Buildings, Borough of Manhattan, New York City.

CINDER CONCRETE IN NEW YORK CITY.

History.—Prior to about 1885, the building laws of New York City allowed the use of practically nothing but burnt clay products as fire-resisting material. The construction of the arch filling or plat-

* Presented at the meeting of March 3d, 1915.

form between steel or iron floor-beams was confined chiefly to heavy solid clay-brick segmental arches of specified dimensions. Gradually, as the general demand for "fire-proof buildings" increased, various digressions were made from the recognized standard floor to include many other forms of porous, semi-porous, or dense terra cotta tile and block, concrete, gypsum, and other mixtures, plain and reinforced in various ways, as well as several odd combinations of all these materials. The law contained a clause which permitted the use of "some equally good fire-proof material" as a substitute for brick arches, but the extremely vague manner in which this phrase was worded, the eager competition existing among manufacturers, and the fact that any new floor system, to be approved, must have been passed upon by the Board of Examiners, of which the membership was largely non-professional, brought about a rather chaotic condition in building affairs.

In order to eliminate the arbitrary method of awarding approvals, and to establish a standard test by which the quality of a fire-resisting floor construction might be judged, Superintendent Stevenson Constable, in 1896, had a number of test ovens erected, and invited leading manufacturers to supply typical specimens of their respective commercial products for the purpose of submitting them to a common test, or tests, which aimed to duplicate as nearly as possible, fire, water, and load conditions obtaining in actual conflagrations. The following is a report* of the test of the chambers and floor systems:

"The test kilns in which this series of tests were made were 11 x 14 feet, inside measurement, and 10 feet high from the upper grates to the floor system, which rested on the walls and formed the roof of the kiln. The walls were 12 inches thick, re enforced with buttresses and metal backstays. There were two sets of grates, the vertical distance between which was from 14 to 18 inches; air was admitted to same through openings in the walls at the ends of the ashpits. Chimneys 15 inches square were placed at each corner of the kilns. The floors tested were constructed as in actual building practice, the various arches were sprung between steel beams and the ceilings plastered as in a finished job. The wood flooring was not laid, but sleepers with concrete filling between them were in every case.

"The method of testing was to place a uniform load of 150 pounds per square foot on the central floor arch, then start a wood fire on the grates and keep it burning for a period of five hours, endeavoring

* *The Engineering Record,* September 18th, 1897.

to keep up a temperature of 2 000 degrees Fahr. for the last four hours. At the expiration of five hours water was applied to the interior of the structure for 15 minutes by the New York Fire Department, through a 1⅛-inch nozzle, under a pressure of 60 pounds per square inch; the first five minutes on the ceiling only, and the remaining 10 minutes on the walls and ceiling, principally the latter; then the top of the floor was flooded by a stream under lower pressure for five minutes. Subsequently the load of 150 pounds per square foot was removed from the central arch, and another of 600 pounds per square foot substituted; this was kept on for 48 hours, which concluded the test.

"The temperatures were taken just below the floor system by the pneumatic pyrometer made by Uehling, Steinbart & Co., of Newark, N. J., and also by placing various metals with known melting points in the kiln, in a position similar to that of pyrometer tube. Three iron rods upon which scales were fastened were placed along the center line of the central (loaded) arch in line parallel to the floor beams. One rod was placed at the center of the span of floor beams and one at each end; these scales on the rods were read by means of a transit giving the combined deflections of both floor beams and the arch between them."

Fourteen different types of fillings between steel beams were tested in this manner,* of which seven consisted essentially of plain or reinforced cinder concrete arches or slabs. In three of the remaining types the load-carrying members depended on this material for a portion of their strength.

From this series of tests it may be seen that the cheapness and lightness of the clinker resulting from the combustion of anthracite coal, used as an aggregate in this special form of concrete construction, would insure its increasing popularity, provided it met with service requirements. No disapprovals based on the results of these tests prohibiting the future use of any of the fourteen systems were issued by the Bureau of Buildings. No action was taken on two of the systems, one of which was of cinder concrete; and approvals were granted to twelve.

In 1902, after an interval of about 4 years, similar tests were conducted on various systems. This was practically coincident with the establishment of the Columbia University Fire Testing Station by Professor Ira H. Woolson. From that time forward the number of patented floors, and the consequent number of tests, increased

* *The Engineering Record*, September 18th, 25th, October 2d, 9th, 1897.

rapidly. In 1903 and 1904 the official tests averaged about one per month, but since that time there has been a gradual decrease in their number, and at present such tests are extremely rare. Exclusive of the 1896 series, 46 approvals have been issued by the Bureau of Buildings, based on fire, load, and water tests which have differed in prescribed severity to some extent from the original requirements. In 27 of these, or in more than one-half, cinder concrete has been the essential structural material of the filling between the steel beams.

It will be noted that an approval of a new floor system, through the agency of a report on a successful fire, load, and water test, permitted the use of the construction in question for spans not exceeding those included in the fire test, and to support safely a live load of 150 lb. per sq. ft. and no more. To obviate the necessity for additional fire tests on a system which had been approved as satisfactory in fire resistance, and for which an increase in span or live load, or both, was desired by the manufacturer, a load test was devised. This consisted in placing on a typical arch, similar to that for which approval was desired, and constructed for the purpose, a uniformly distributed load consisting of bags of sand, bricks, pig iron, paving blocks, or any other available material. This material was added until the treatment caused the total destruction of the arch, or until the desired load was attained. Approval was then granted for a safe live load per square foot equal to one-tenth of the maximum load per square foot placed on the arch. This method was used first in 1896 on an isolated section of one of the floor fillings after it had been subjected to a fire and water test. Bricks were used for the load, in this case, and were stacked in a solid mass. In fact, photographs of the earlier test loads are evidence of the extensive use of this material, as well as bags of sand or cement, and paving blocks stacked in a mass, interlocked when possible, with little, if any, indication of any attempt to prevent the consequent arching of the superimposed load. (Fig. 1.) Later, cast-iron pigs, steel billets, or similar material, cribbed and generally interlocked, were stacked in many cases to enormous heights, and slabs were approved for working loads, the accuracy of which is extremely doubtful. (Fig. 2.) These features are discussed quite fully in a recent paper* by Guy B. Waite, M. Am. Soc. C. E. A definite attempt to obtain greater accuracy was shown in the later

* "Cinder Concrete Floors," *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 1773.

FIG. 1.—LOADING WITH BAGS OF SAND. NOTE THE LONGITUDINAL SCANTLING UNDER TIMBERS, AND THE UNLOADED OR ANCHOR-SPANS OF CONCRETE, SOLID THROUGHOUT THE ENTIRE DEPTH OF I-BEAMS.

FIG. 2.—LOADING WITH PIG IRON STACKED IN A SOLID MASS. NOTE EVIDENCES OF ARCH ACTION AND CONSEQUENT TRANSFER OF LOAD TO SUPPORTING I-BEAMS.

FIG. 1.—LOADING WITH BAGS OF SAND. NOTE THE LONGITUDINAL CRIBBING UNDER TIMBERS, AND THE [illegible]

FIG. 2.—LOADING WITH PIG IRON ARRANGED IN A SOLID MASS. NOTE [illegible]

Fig. 3.—Loading with Segregated Piles of Pig Iron. Note that the Stacks Have Come into Contact, Due to Deflection of Slab.

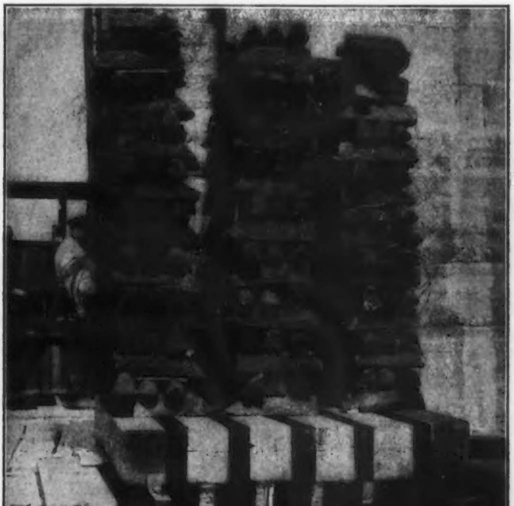


Fig. 4.—Loading with Segregated Piles of Pig Iron. Note the Dangerous Condition of the Stacks.

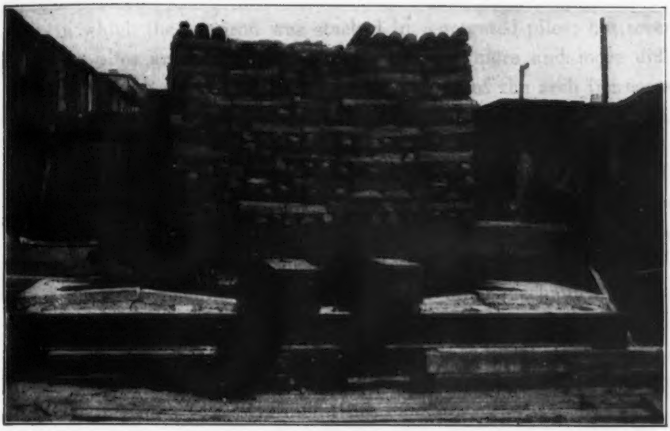


FIG. 5.—LOADING AT THE ONE-THIRD POINTS OF SPAN. ACCURATE AND COMPARABLE RESULTS ARE POSSIBLE WITH THIS METHOD.

FIG. 6.—CONDITION OF 5 FT. 3-IN. FLOOR-SLAB OF 1:2:5 CINDER CONCRETE AT COMPLETION OF A STANDARD 4-HOUR FIRE TEST. SUPPORTING THIS IS SHOWN THE 12-IN. SIDE-WALL OF THE TEST STRUCTURE, ALSO OF CINDER CONCRETE.

Fig. [illegible]

Fig. [illegible]

tests, in which the pig iron was stacked in segregated piles; but, even when the piles are carefully placed, it becomes more and more difficult, as the loading progresses and the deflection of the arch increases, to continue without causing the piles gradually to come together at the tops. (Figs. 3 and 4.) It was only recently that this method was superseded in the Building Bureaus by one in which the load is applied by two equal concentrations symmetrically placed at the one-third points of the span. (Fig. 5.) This innovation simplifies the work of conducting a conclusive test, and renders possible the use of a testing machine, a hydraulic jack, or other mechanical means. It also obviates the necessity of applying actual weights, with the attendant danger to the laborers and the expenditure of time. To cause the failure of many of the test specimens, it often became necessary to handle excessive quantities of material.

As the earlier tests were along the line of investigative work in a new field, and were designed to meet immediate needs, one might expect the procedure and methods to be rather crude and faulty. The tests, however, served as a working basis for the earlier application of cinder concrete.

Of the 193 existing approvals in the Manhattan Building Bureau which are based on the fire, load, and water tests in conjunction with the special loading tests, 133 were issued for constructions consisting of reinforced cinder concrete.

Fire Resistance.

To any one at all familiar with the effect on cinder concrete of flames at high temperatures followed by the application of cold water at high pressure, little other evidence is necessary to prove its efficiency under this treatment. Experience is not confined merely to the results noted at the completion of standard tests, but is also based on examinations of buildings after having passed through actual conflagrations. The typical appearance of this material (mixture 1:2:5) after suffering the maximum punishment from fire and water may be seen clearly in Fig. 6, which is reproduced from a photograph taken from a point directly under and looking upward at the under surface of a floor slab which had passed through the standard tests, conducted by one of the writers. The portion of the wall, it may also be noted, forms one of the 12-in. side or enclosure-walls

of the test chamber. The fire test, at 1 700° Fahr., lasted 4 hours, and was followed by the application of water for about 5 min., which produced the slight pitting noted on the slab surface. The wall, Fig. 7, on the other hand, had been subjected to seven such tests, aggregating 28 hours of fire.

Pitting on the surface of cinder concrete is due chiefly to the combustion of small particles of exposed unburned coal, in addition to the effect of de-hydration produced on all concrete. One of the chief arguments, derogatory to cinder concrete as a fire-resisting material, is that unburned coal, occurring in the aggregate in appreciable quantities, when burned out, will produce flaws and honeycombing, and consequent loss in strength. Portions of the slab shown in Fig. 6 were removed and inspected carefully. It contained a rather large percentage of unburned coal. At the very surface of the slab, where the flames had played directly on it, the coal had carbonized to a greater or less extent, but the body of the concrete remained practically intact, as shown by Fig. 8. In many instances pieces of hard, sharp, unburned anthracite were found within ½ in. of the exposed surface. The appearance of the fractured cross-section of this slab, on the other hand, seemed to indicate that actual de-hydration or other disintegration, as shown by the discoloration of the concrete, had occurred to the depth of from 1 to 2 in. of the original surface. Also, at one point there was a piece of charred wood, almost entirely consumed, which, judging by the size of the void in which it was found, had evidently been a part of a common lath. This void was about 2¼ in. from the surface which had been subjected to fire. This may be seen on Fig. 9, which also shows the location of a piece of brown paper which appeared to be slightly charred at its edges. A temperature high enough to anneal the cold-drawn wire reinforcement was certainly reached, as indicated by the reduction in its tensile strength and corresponding change in its elastic properties. Specimens of wire taken from the haunches and other unexposed locations showed a tensile strength of 74 100 lb. per sq. in., and this was reduced to 47 000 lb. per sq. in. for samples taken from the exposed portion of the slab proper. This reinforcement was placed 1 in. up from the bottom side of the slab. That the main body of the concrete possessed its original strength was evident from the fact that, as a result of actual tests on samples taken from the unexposed

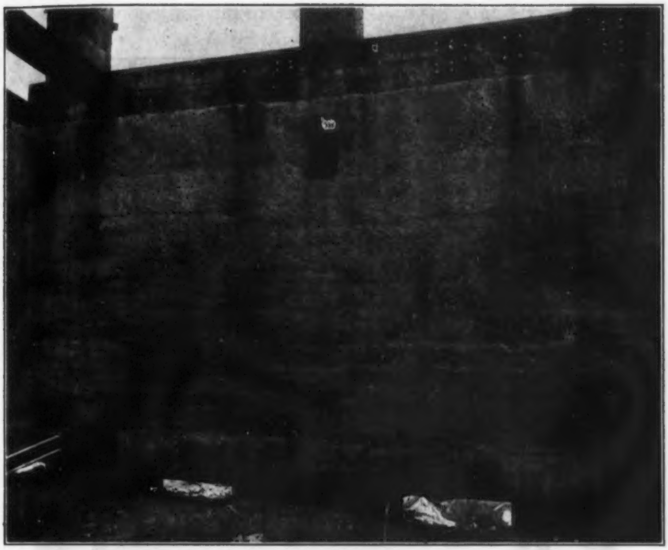


FIG. 7.—BACK-WALL OF STANDARD TEST HOUSE. THIS WALL WAS SUBJECTED TO 28 HOURS OF FIRE, AND NUMEROUS APPLICATIONS OF WATER UNDER PRESSURE.

FIG. 8.—PORTIONS OF SLAB SHOWN IN FIG. 6. UPPER PART SHOWS UNDER OR EXPOSED SIDE OF SLAB; LOWER PART SHOWS CROSS-SECTION.

FIG. 9.—PORTIONS OF SLAB SHOWN IN FIGS. 6 AND 8. VERTICAL SCALE INDICATES DISTANCE UP FROM ORIGINAL UNDER SIDE OF SLAB.

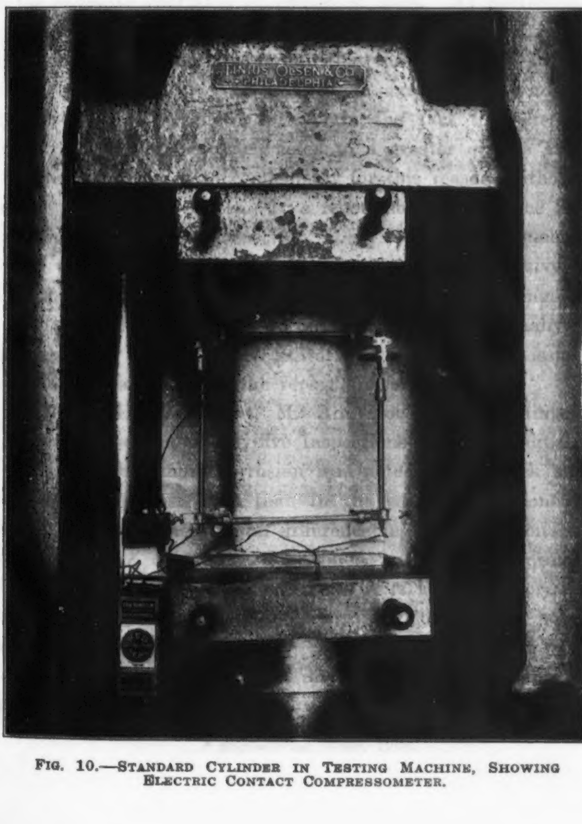


FIG. 10.—STANDARD CYLINDER IN TESTING MACHINE, SHOWING ELECTRIC CONTACT COMPRESSOMETER.

haunches and from the exposed slab, cubes of the former gave a compressive strength of 1 182 lb. per sq. in. normal to bedding, as against 1 273 lb. per sq. in. for cubes of the latter. Allowing for the shape of the test piece, and reducing the value obtained with cubes to that which might be expected with prisms having a vertical dimension equal to twice their width, it is found that the material subjected to fire had a compressive strength of 1 120 lb. per sq. in., and that material not thus treated showed a strength of 1 040 lb. per sq. in.

Corrosion.

Until recently, there has been much agitation relative to the question whether or not cinder concrete as such is directly responsible for the excessive corrosion of embedded steel. No particular reason was advanced as conclusive affirmative evidence of the truth of this idea, except the popular assumption that the sulphur in the aggregate became extremely active as a corrosive agent. For some time no other theory was advanced to explain the fact that steel or iron embedded in cinder concrete rusted to a greater or less extent, and it became rather a fad to condemn it for this reason, without intelligent inquiry into the causes. In a report submitted to the Structural Association of San Francisco, Mr. C. F. Wieland recommended that the use of cinder concrete be prohibited, due to "corrosive qualities and lack of strength". The doubtful wisdom of condemning, generally, a product which, within proper limitations, possessed many valuable characteristics, was criticized editorially in the same issue of the technical journal in which the report appeared.*

A. L. A. Himmelwright, M. Am. Soc. C. E., claimed,* from experience gained in twenty-five inspections made during alterations of buildings, that more corrosion was noted where brick and stonework abutted against steel than in the case of any concrete. He also observed that there is no difference in the protection offered by stone or cinder concrete. Segregated sulphur, according to Mr. Himmelwright, is not possible, because it is largely vaporized during combustion, thus rendering the uniformly distributed sulphur extremely small.

Professor Charles L. Norton draws the following conclusions, in regard to the corrosion of steel in cinder concrete, as a result of

* *Engineering News*, 1906.

his well-known and widely-quoted series of experiments for the Insurance Engineering Station:

"Sulphur might [cause corrosion], if present, were it not for the presence of the strongly alkaline cement; but with that present the corrosion of steel by the sulphur of cinders in a sound Portland concrete is the veriest myth, and as a matter of fact the ordinary cinders, classed as steam cinders, contain only a very small amount of sulphur. There can be no question that cinder-concrete has rusted great quantities of steel, but not because of its sulphur, but because it was mixed too dry, through the action of the cinders in absorbing moisture, and that it contained, therefore, voids; and secondly, because in addition the cinders often contain oxide of iron which, when not coated over with the cement by thorough wet mixing, causes rusting of any steel which it touches.

"There is one cure and only one, mix wet and mix well."

The tests conducted by Mr. William H. Fox* at the Thayer School of Engineering, which were designed in a measure to throw light on the matter of corrosion, should not, in the writers' opinion, be considered at all conclusive. Primarily, the method of mixing the concrete was quite dissimilar to that common in building practice. The procedure consisted of spreading the cinders on a flat surface, with all the valuable void-filling fine material removed, and adding water to them until "no more was taken up". The dry sand and cement were then thrown on the wet cinders and the whole mixed together. It may readily be imagined that this manner of preparation of ingredients was not conducive to the making of dense or even typical concrete. The results, therefore, should be examined with this detail in mind. The variables introduced were, mix, consistency, method of tamping, and treatment during aging. Some specimens were cast dry and tamped, others wet and untamped, and still others wet and tamped. Several mixes, varying in richness from 1: 1: 3 to 1: 3: 6, were cast. Alternate water and air, or steam and air, curing were the treatments used. Steel of various shapes, embedded to a depth of 2 in. in all directions, corroded to a greater or less extent, regardless of the consistency, mixing, or method of casting the encasing concrete. Mr. Fox's conclusions are as follows:

"To secure a dense homogeneous cinder concrete, a thorough tamping is necessary. A rich mixture, either 1: 1: 3 or one in which the proportion of cement to aggregate is larger, should be used in all

* *Engineering News*, May 23d, 1907, p. 569.

cases. The greatest of care should be taken in mixing the materials, and it may be necessary to resort to the seemingly impractical method of coating the reinforcement with grout before placing in the concrete."

Mr. Fox also admits the severity of the tests.

As an exception to general experience, an instance of the effectiveness of porous cinder concrete was noted during the demolition of the Pabst Hotel,* New York City. It was there noted that corrosion of the steelwork existed only to a superficial extent. This fact becomes a matter of special interest when the type of floor system there used is considered in detail. The practice of the manufacturer of the arches in question was to deposit the concrete by throwing it from shovels on the wire mesh centering, omitting all tamping and stirring, the object being to obtain a porous product. This, it was claimed, would make it safe to deposit it in freezing weather, due to the absence of superfluous water. At the time of removal this construction had been in place 8 years.

Not so long ago, an instance of the inefficiency of porous cinder concrete used in a roof slab was brought before the public by the necessity for the removal of the roof of the train-shed of the La Salle Street Station, Chicago, Ill. In this case the cinder concrete, protected by a thin shell of stone concrete, reinforced with expanded metal, and exposed to gases and moisture coming from below, had to be renewed because of excessive corrosion.

Additional information relating to the corrosive tendencies of cinder concrete may be found in a series of articles and references listed in "A Bibliography of Corrosion of Iron and Steel in Cinder Concrete."†

An examination was made of the condition of the cold-drawn galvanized wire reinforcement, contained in the slab previously described, as to fire-resisting qualities. The floor system, after the completion of the fire, load, and water tests, had been directly exposed to the elements for more than a year without protection of any kind other than a 2-in. fill. In the region where the fire and water had pitted the surface, the wires, of course, were corroded. Where the concrete and reinforcement were still in practically their original condition, that is, in the haunches and anchorages, the wire was in much better condition. In many cases, long stretches of wire

* Report of Engineering Staff, Bureau of Buildings, New York City; also in Vol. 49 of *Engineering News*.

† *Engineering News*, April 18th, 1912.

were only affected slightly, despite the undue severity of the exposure. Corrosion was most apparent at the junction points where the lateral wires crossed the longitudinals, and seemed generally to have its beginning in the pocket or crevice thus formed in the concrete.

It is the writers' opinion that the corrosion of steel embedded in cinder concrete is still a matter of conjecture. The original reason for its existence was due, in all probability, to the idea prevailing, formerly, that concrete must be placed dry and religiously tamped and rammed in order to attain maximum density.

Present-day knowledge has led practice in many cases to the opposite extreme, with the result that concrete is often deposited too wet; but, cinder concrete, due to the very form of the aggregate, should not be deposited dry. By giving the well-mixed material a wet and viscous consistency, and constantly stirring it, the mortar is able to coat the reinforcement thoroughly and flow readily into the interstices of the larger clinkers, a condition which cannot obtain with dry material, no matter to what extent it is rammed. This practice, more than anything else, will tend toward the reduction of corrosion, which should not exist to any greater degree in cinder than in stone concrete, if proper care is exercised. This opinion is based on experience gained with the use of anthracite coal cinders. Western practice seems to indicate that, with the use of bituminous coal cinders as an aggregate, extreme and unexplained cases of corrosion have occurred.

Strength and Other Characteristics.

Cinder concrete as used in New York City is made with ingredients of widely differing characteristics. Materials used as the coarse aggregate vary by all degrees from the hard, vitreous clinker classed as "steam cinders" resulting from the combustion of anthracite coal, which is often free from ash and other foreign matter, to the almost useless, soft, flaky ash such as is commonly obtained from domestic coal stoves. Due primarily to the impossibility of adequate inspection, arising from the lack of a sufficient number of officials engaged in these duties, the resulting concrete is about as variable in quality as the available ingredients.

To make proper allowance for the variableness of the aggregate and for the workmanship, a factor of safety of 10 has been required

by the Building Bureau authorities in all load tests on cinder concrete floors.

The doubtful economy of introducing such a large factor of safety on selected test material, with resulting wide variation in the commercial product, should militate against a continuance of this method of control. The question then arises whether or not the product is adaptable to more scientific regulation, that is, can it be safely designed? An answer to this question can be found only in experience and knowledge gained by systematic research.

Present Investigation.

This absence of accurate knowledge of cinder concrete has led to the inception of an investigation, together with a series of experiments by the Department of Civil Engineering of Columbia University, partly in co-operation with the Bureau of Buildings, Borough of Manhattan, New York City.

General Scheme.—The investigation, instituted during the spring of 1913 and now in progress, is based on the following general scheme:

1.—The determination of the compressive strength, with elastic and other properties, of typical New York cinder concrete, cast under working conditions, as obtained in actual practice;

2.—A study of the behavior of typical cinder concrete slabs under load, as affected by the character and percentage of reinforcement, thickness, span length, age, and local conditions, for the purpose of aiding in theoretical design or establishing a standard loading test;

3.—An attempt to recommend specifications tending toward an improvement in the product, that is, greater density, uniformity, etc.

Typical New York Material.

Tests for compressive strength and elastic properties were conducted on standard cylinders, 16 in. high and 8 in. in diameter, as recommended by the Joint Committee. Galvanized-iron forms, having these interior dimensions, were taken to buildings in course of construction in various parts of Manhattan, and 40 cylinders were cast at each location.

All specimens were made by men familiar with the casting of test cylinders, and under the personal supervision of a Building Bureau official. The mixture in the specimens was identical with

that going into the floors at the time of arrival, no advance notification having been sent to the contractor of the intention to take samples. The various methods of handling, tamping, etc., were carefully noted. Test pieces were taken from five buildings selected from three arbitrarily chosen zones, the intention being to find local variations, if such existed. Samples of the component materials were selected from the cement, sand, and cinders, in each case, and tests and analyses of these were made. The writers wish at this point to acknowledge the cordial co-operation of Mr. J. F. Davis, of Robert W. Hunt and Company, in furnishing chemical analyses of all materials used in this investigation.

The cylinders were allowed to remain for several days under the atmospheric conditions obtaining on the mixing floors of the buildings in which they were cast, after which time they were removed to the Columbia Laboratories and stored in air at ordinary room temperature, until tested. During the early aging they were sprinkled occasionally. It was thought that this method of storage duplicated as nearly as possible the conditions under which the floor slabs themselves are cured. In many cases, due to their thinness, the slabs dry out quite rapidly. All cylinders were faced at both ends with plaster of Paris, before testing.

From each of the sets of 40 cylinders 10 were tested respectively, at 1, 2, 6, and 12 months. Fig. 10 shows a standard cylinder in the testing machine. From Table 1, showing the results of these tests, data on material from Building No. 5 have been purposely omitted. The latter was one of the new Columbia dormitory buildings, then in process of construction, and the contractor was aware of the intention to select samples. These data are in Table 12.

Zone *A* applies to that region of Manhattan occupied for the most part by apartment houses.

Zone *C* is the down-town office building region.

Zone *B* refers to that large section lying midway between Zones *A* and *C*, and in which the new construction consists mainly of loft and mercantile buildings.

The arbitrary choice of zones was made, not only for the purpose of obtaining material from different types of buildings in process of construction, but also to study the effect of the conditions occurring at the source from which the cinders were obtained; for, in many

cases, the existence of oil, injurious chemicals, certain by-products of manufacture, and other impurities are important factors detrimental to the resulting concrete.

TABLE 1.—DATA RELATING TO CINDER CONCRETE AS USED IN FIRE-PROOF FLOORS IN NEW YORK CITY.

Zone	A_1	B_1	B_2	C
Mix	1 : 2 : 5	1 : 1 : 5	1 : 2 : 5	1 : 2 : 5
Method	Continuous mixer. Coltrin.	By hand turned twice.	Batch mixer.	Mixer, Ransome.
Cement	Alsen.	Dragon.	Vulcanite.	Atlas.
Sand	Typical Long Island Bank Sand, North Shore.			
Cinders	Anthracite. Ice plant.	Local hotel steam plant.	Local.	Local office building steam plant.
Weight, in pounds per cubic foot	107	100	107	109
One-month test:				
Crushing strength, in pounds per square inch	407	507	818	980
Modulus of elasticity, in pounds per square inch	924 600	857 400	1 230 000	1 492 000
Two-month test:				
Crushing strength, in pounds per square inch	701	662	1 254	1 035
Modulus of elasticity, in pounds per square inch	1 134 000	1 030 000	1 740 000	1 428 250
Six-month test:				
Crushing strength, in pounds per square inch	933	754	1 744	1 478
Modulus of elasticity, in pounds per square inch	971 000	1 050 000	1 348 000	1 276 000
One-year test:				
Crushing strength, in pounds per square inch	913	813	1 465	1 475
Modulus of elasticity, in pounds per square inch	993 000	956 000	1 200 000	1 320 000

Each value in Table 1 is an average obtained from 10 samples, except in the case of B_2, in which the total number of specimens tested was 30 instead of 40. This was due to the fact that about three-quarters of the material used was mixed under actual working conditions, and the remaining 10 cylinders were cast from a mix prepared by the contractor's foreman, who was aware of the use to which the test specimens were to be put. His evident zeal in the matter resulted in extremely high compressive strengths, and therefore these were excluded from the averages.

The modulus of elasticity was determined at a point on the elastic curve corresponding to one-fourth the ultimate strength.

The writers believe that Table 1 contains sufficient data to prove conclusively the variableness of cinder concrete as used at present in Manhattan. The strength of Sample A, 407 lb. per sq. in. at an age of 1 month, may be taken as typical of concrete resulting from the use of low-grade cinders, despite the fact that it was machine-mixed.

B_1 was found to have a strength of 507 lb. per sq. in., after a similar interval of time. In this case, the use of good cinders and a richer mix than is required did not offset the poorer results almost always obtained by ordinary hand-mixing. Sample B_2 approaches, and C about equals, good practice in cinder concrete construction at the present time. Fig. 11 shows fractured cylinders cast from typical New York materials.

Samples B_1, B_2, and C represent cinders of approximately the same gradation in size as reported in Table 6, Appendix I. The complete mechanical analyses of both the typical concrete aggregate and the test slab aggregate are given in Tables 6 and 9. The mechanical analysis of Sample A shows a greater preponderance of fine material in this cinder than in the other three samples. The conclusion as to poorer results being obtained with hand-mixing appears to be justified in view of the detailed tests of the component materials reported in the Appendices.

Behavior of Typical Slabs Under Load.

To date, tests have been conducted on 58 slabs, together with tests on corresponding cylinders and component materials.

Component Materials.

Cement.—The cement used in the 1913 series was the Dragon Portland delivered to the contractor for the fire-proof floor construction of the new dormitory building, Furnald Hall, Columbia University. In the 1914 series the cement used was Dragon Portland furnished through the courtesy of the Lawrence Cement Company.

The results of the detailed tests are given in Appendix II.

Sand.—The sand used in both instances was the typical bank material obtained from the north shore of Long Island. Its physical properties are given in Appendix II.

Cinders.—For the 1913 series, the cinders were a typical commercial product, furnished, by a dealer in this material, to the contractor for the floors of Furnald Hall.

The cinders for the 1914 series were obtained from the Columbia University power plant.

The physical properties of both aggregates appear in Appendix II.

Reinforcement.—The steel reinforcement consisted of triangular

FIG. 11.—FRACTURED CYLINDERS CAST FROM TYPICAL NEW YORK MATERIALS.
A.—POOR MATERIAL CONTAINING LARGE PERCENTAGE OF DUST AND LARGE CLINKERS.
B_1.—FAIR MATERIAL; BUT CONCRETE IS POROUS.
B_2.—GOOD CONCRETE.
C.—GOOD CONCRETE.

FIG. 12.—CASTING CYLINDERS AND SLABS, 1913 SERIES, DORMITORY BUILDING, COLUMBIA UNIVERSITY.

wire mesh generously furnished by the American Steel and Wire Company, expanded metal kindly submitted by the Expanded Metal Engineering Company, and the following types obtained in the open market: Electrically welded wire mesh, triangular wire mesh, expanded metal, plain round, plain square, and twisted square, rods.

TABLE 2.—SCOPE OF TESTS ON SLABS.

Concrete.			Number of Specimens.						
			Slabs (All 22 in. wide).						
Mortar.	Aggregate.	Consistency.	Restrained.	Simple.	Span, in feet	Thickness, in inches.	Cylinders, 8 by 16 in.	Reinforcement.	Variables.
1913 Series									
Dragon cement. Typical Long Island sand.	Commercial cinders.	Wet.	6	10	6	4	20	Triangular wire mesh. Welded wire mesh. Expanded metal.	Span; Thickness; Condition of end restraint; Percentage and character of reinforcement.
		Wet.	6	6	8	4	15		
		Medium.		3	8	5			
1914 Series									
Dragon cement. Typical Long Island sand.	Commercial cinders from Columbia University steam power plant.	Viscous.	3	3	7	4	10	Triangular wire mesh. Expanded metal. Round rods. Square rods. Twisted square rods.	Span; Thickness; Condition of end restraint; Percentage and character of reinforcement.
		do.	3	3	6	4	10		
		do.		14	5	3 and 4	10		
		do.	(1—Special.)		7	4			

Total cylinders 65

Restrained	18
Simple	39
Special	1
Total number of slabs	58

Concrete.—The concrete in all cases consisted of 1 part Dragon cement, 2 parts Long Island sand, and 5 parts steam cinders, all by volume.

For the 1913 series, the materials were brought to a continuous mixer, mixed for about 1 min., deposited in wheel-barrows, transported about 200 ft., and dumped into the forms provided for the purpose,

and following throughout a procedure exactly similar to that used in casting the actual floors of the building. Fig. 12 shows the slabs and cylinders in process of construction. The concrete was furnished through the courtesy of the Department of Buildings and Grounds, Columbia University. The consistency of the concrete was wet in some slabs and medium in others.

The materials for the 1914 series were mixed in the ½-cu. yd. Blystone batch mixer of the Columbia Concrete Laboratory, which is used for most of the investigative and commercial work of this kind. The cement, sand, and cinders, carefully measured by volume, were deposited dry in the machine. Water from a hose connecting with a measuring tank was played on the ingredients as the mixing went on, the percentage of water being practically constant throughout the series. The process of mixing occupied about 2 min. per batch. The concrete was taken to the forms in wheel-barrows, and when deposited was of a wet, viscous consistency. It was stirred and pushed into the spaces between the wires or rods with narrow pieces of scantling, the material being too wet to permit of tamping, as the term is generally understood. An excellent, dense, homogeneous concrete was thus obtained. The casting of restrained and simple slabs is shown in Figs. 13 and 14.

The slabs for the most part remained in the room in which they were cast until the time of test, and were sprinkled occasionally during the early periods of aging. Cylinders cast at the same time received similar treatment.

METHOD OF CONSTRUCTION.

The condition of end restraint and other details of construction of that type of arch styled in this investigation as "Restrained" may be best understood from an examination of Figs. 13, 15, and 19. In this construction wire mesh reinforcement was used exclusively, the slabs being in all cases 22 in. wide. The "free" slabs, or simple beams, all 22 in. wide, were cast as shown in Fig. 14. The uniform width of 22 in. was assumed in order to facilitate the testing of the simple beams in a testing machine, this being the maximum width possible, due to the spacing of the screws. Arches commonly known as "flat ceiling" or "bottom flange construction" have not yet been investigated.

Fig. 13.—Casting "Restrained" Slabs, 1914 Series.

Fig. 14.—Casting "Simple" Slabs, 1914 Series.

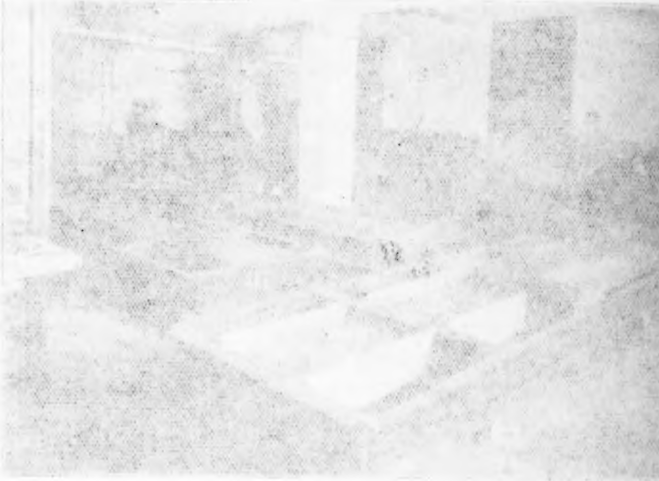
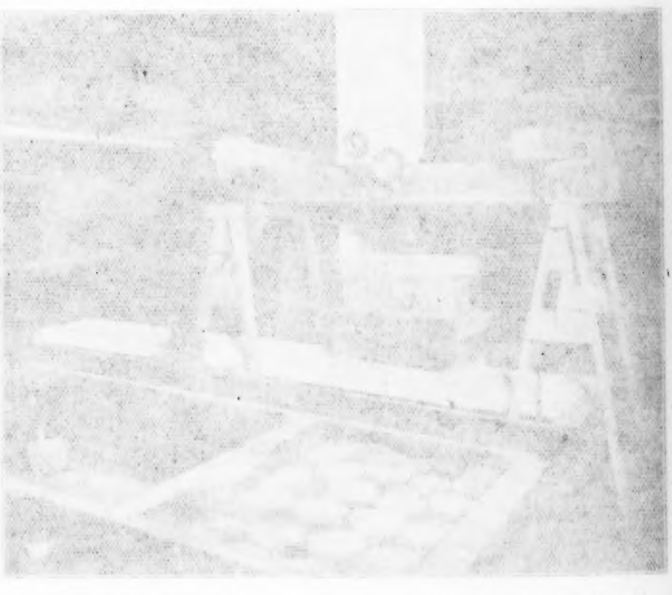


FIG. 15.—TESTING "RESTRAINED" SLAB WITH HYDRAULIC JACK. NOTE GAUGES AND RECORDING INSTRUMENTS IN PLACE.

FIG. 16.—TESTING "SPECIAL TYPE", REPRESENTING END-BAY CONSTRUCTION.



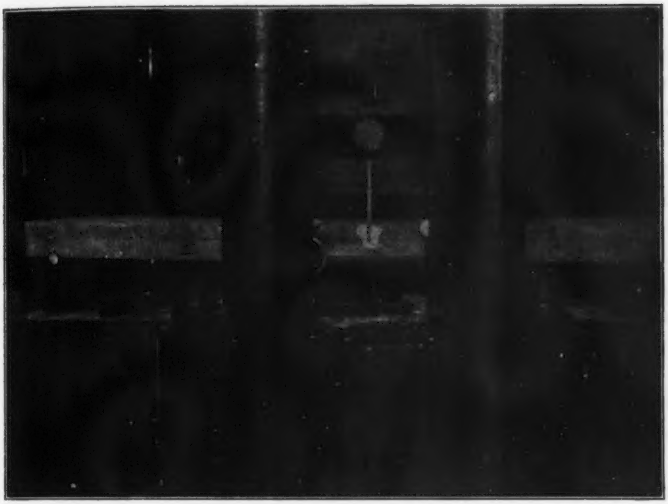


Fig. 17.—Loading "Simple" Slab in Laboratory Testing Machine.

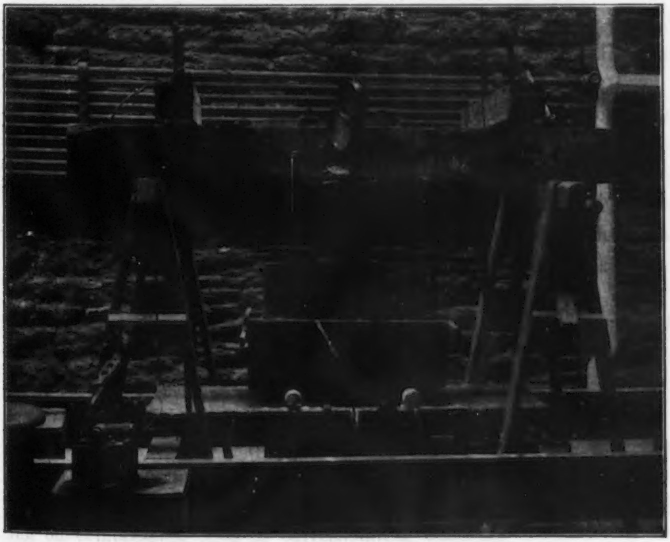


Fig. 18.—Testing "Simple" Slab with Hydraulic Jack.

Methods of Testing.

Fig. 15 shows clearly the arrangement used in loading all the restrained arches. A hydraulic jack and hand-pump were used in applying the load, the latter being indicated on the gauges shown. The jack and attached gauges had been previously calibrated as a unit in a testing machine of known accuracy. The load was applied in increments of 5 lb. gauge pressure, corresponding to about 250 lb. of actual load.

The "Special" type mentioned in the table of results (Table 3) is shown in Figs. 16 and 19, the loaded span representing conditions obtaining in actual construction at a point where the slab abuts against an outside wall, or an enclosure of a vertical opening, and where proper end restraint is lacking.

Simple slabs were tested, in part, as shown in Fig. 17, resting on rollers placed on the platen of a testing machine or by the jack, as in Fig. 18.

Deflections were read at each side to the nearest hundredth of an inch by a steel scale, set in plaster of Paris at the mid-point of the span, intersected by a wire stretched over nails driven into the sides of the slab, at points directly over the roller supports in the case of simple slabs, or directly over the flanges of the supporting I-beams in the case of restrained slabs.

The strain produced on the tension side of the slabs was measured on 10 or 12-in. gauge lengths between two points symmetrically placed in the surface of the slabs, in the plane of the reinforcing steel which, in all cases, was cast 1 in. up from the bottom surface of the arch. In the case of restrained beams, the reinforcement was turned up at the one-fifth point of the span, in order to pass over the supporting I-beams.

Compressive strain was measured on a 10 or 12-in. gauge length on the sides of the slabs, ½ in. down from the top surface of the arch, and afterward corrected for the corresponding probable extreme fiber strain.

Tensile and compressive stresses were computed from these measured strains, assuming the modulus of elasticity of steel to be 30 000 000 lb. per sq. in., the proper modulus for the concrete depending on compression tests of corresponding material.

Results.

Table 3 is a statement of the results of tests to date on slabs.

Computations and Discussion.

Form of Test Specimens and Applied Load.

Previous to the inception of the present series of tests, a thorough study was made of all records of load applications on cinder concrete

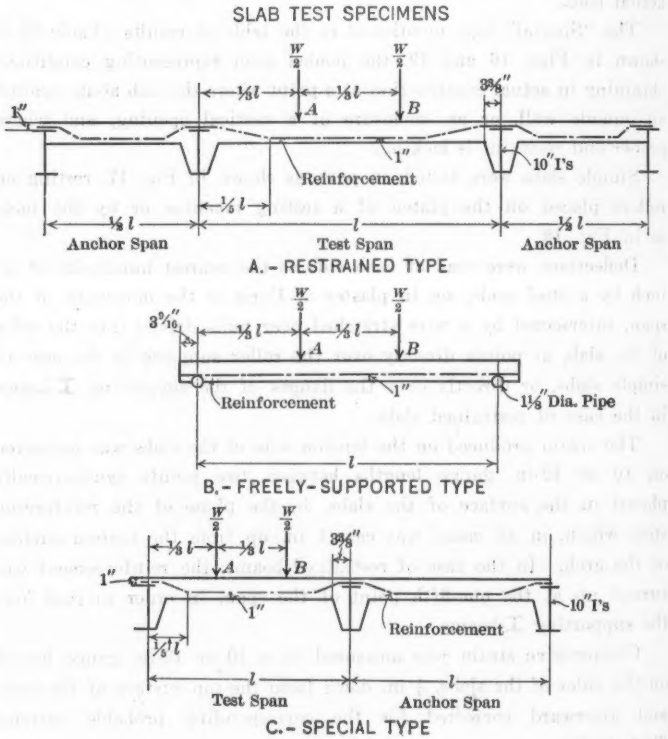


Fig. 19.

floor filling between steel girders; and all such tests that came under the direct observation of the writers were carefully inspected. In all the tests conducted during the years prior to 1911, the mode of installation, as regards anchorage of the reinforcement and provision

TABLE 3.—Results of Slab Tests.

Slab No.	Reinforcement.	Span.	Thickness. Net.	Thickness. Gross.	End condition.	Percentage of steel.	Age, in weeks.	(jd)	(kd)	(n)	Maximum applied load, in pounds.	Maximum deflection, in inches.	Estimated weight, in pounds.	Equivalent load, in pounds per square foot.	Uniform dead load, in pounds per square foot.	Equivalent total uniform load, in pounds per square foot.
C 7-ft.																
1	Triangular wire mesh. Area, 0.173 in.	7′ 1″	3.16	4.16	R.	0.26	4	2.85	0.98	26	8 700	1.22	481	894	37.0	931
2		7′ 1″	3.08	4.08	R.	0.27	4	2.77	0.95	26	8 175	1.70	473	839	36.4	875
3		7′ 1″	3.16	4.16	R.	0.26	4	2.85	0.98	26	7 475	1.86	481	768	37.0	805
7-ft.																
1	" " " "	7′ 1″	3.13	4.13	F.	0.26	4	2.83	0.97	26	2 900	1.78	478	298	36.8	335
2		7′ 1″	3.53	4.33	F.	0.23	4	3.20	1.08	26	2 650	1.63	525	276	40.4	316
3		7′ 1″	3.33	4.33	F.	0.25	4	3.00	1.00	26	2 750	2.15	501	283	38.6	322
7-ft. Special.	" " " "	7′ 1″	3.00	4.00	F. & R.	0.27	4	2.70	0.93	26	4 675	0.63	463	481	35.6	517
C 6-ft. 1		6′ 0″	2.90	3.90	R.	0.28	4	2.61	0.84	22	8 675	1.25	386	1 051	35.1	1 [illegible]
2	" " " "	6′ 0″	2.97	3.97	R.	0.28	4	2.67	0.86	22	10 075	1.55	[illegible]	1 220	35.7	1 256
3		6′ 0″	3.27	4.27	R.	0.25	4	2.95	0.95	22	10 920	0.89	423	1 325	38.5	1 364
6-ft. 1		6′ 0″	3.02	4.04	F.	0.27	4	2.72	0.88	22	3 750	1.97	400	455	36.4	491
2	" " " "	6′ 0″	3.40	4.40	F.	0.24	4	3.06	0.99	22	3 550	1.31	436	431	39.6	471
3		6′ 0″	3.05	4.05	F.	0.27	4	2.75	0.89	22	3 900	1.72	401	473	36.5	510
6-ft. 1 yr.																
1	" " " "	6′ 0″	3.21	4.21	F	0.26	4	2.92	0.87	20	4 138	1.50	425	502	38.6	541
2		6′ 0″	3.02	4.02	F.	0.27	4	2.75	0.82	20	3 800	1.61	406	461	36.9	501
5-ft. Am.																
1	" " " "	5′ 0″	3.20	4.20	F.	0.26	4	2.88	0.99	26	3 875	0.72	346	567	37.6	605
2		5′ 0″	3.25	4.25	F.	0.25	4	2.93	1.01	26	3 050	0.44	351	443	38.2	481
5-ft. Am. 4-in.																
1	" " " "	5′ 0″	2.20	3.20	F.	0.38	4	1.93	0.79	26	950	0.82	264	138	28.7	167
2		5′ 0″	2.05	3.05	F.	0.40	4	1.80	0.74	26	2 215	0.81	252	322	27.4	349
5-ft. Ex.																
1	Expanded metal. Area, 0.180 in.	5′ 0″	3.70	4.20	F.	0.15	4	3.40	0.89	26	2 690	0.55	346	391	37.6	429
2		5′ 0″	3.70	4.25	F.	0.15	4	3.40	0.89	26	3 005	0.33	351	487	38.2	475

TABLE 3.—(*Continued.*)

Slab No.	Reinforcement.	Span.	Thickness. Net.	Thickness. Gross.	End condition.	Percentage of steel.	Age, in weeks.	(*jd*)	(*kd*)	(*n*)	Maximum applied load, in pounds.	Maximum deflection, in inches.	Estimated weight, in pounds.	Equivalent load, in pounds per square foot.	Uniform dead load, in pounds per square foot.	Equivalent total uniform load, in pounds per square foot.
13-C-8																
1	Triangular wire mesh. Area, 0.173 in.	8′ 0″	3.10	4.10	R.	0.26	5	2.82	0.84	20	5 389	2.39	566	490	38.6	529
2		8′ 0″	3.07	4.07	R.	0.27	9	2.79	0.80	17	4 875	2.34	562	444	38.3	482
3		8′ 0″	3.05	4.05	R.	0.27	9	2.76	0.79	17	6 150	1.18	559	559	38.1	597
4		8′ 0″	3.00	4.00	R.	0.27	9	2.73	0.78	17	4 538	1.25	552	413	37.6	451
5		8′ 0″	3.00	4.00	R.	0.27	9	2.73	0.78	17	5 039	1.97	552	458	37.6	496
6		8′ 0″	2.97	3.97	R.	0.28	9	2.70	0.77	17	6 150	3.07	548	559	37.4	596
13-8																
1	" " " "	8′ 0″	3.00	4.00	F.	0.27	10	2.73	0.78	17	2 750	2.65	552	250	37.6	288
2		8′ 0″	3.00	4.00	F.	0.27	10	2.73	0.78	17	2 970	4.28	552	270	37.6	308
3		8′ 0″	3.00	4.00	F.	0.27	10	2.73	0.78	17	2 750	3.20	552	250	37.6	288
4		8′ 0″	3.12	4.12	F.	0.26	10	2.84	0.81	17	2 020	1.35	569	184	38.8	223
5		8′ 0″	3.00	4.00	F.	0.27	10	2.73	0.78	17	2 520	5.43	552	229	37.6	267
6		8′ 0″	3.25	4.25	F.	0.25	10	2.96	0.85	17	2 520	3.40	587	229	40.0	269
13-8.5-in.																
1	" " " "	8′ 0″	4.00	5.00	F.	0.21	10	3.68	0.92	17	3 470	5.22	691	316	47.2	363
2		8′ 0″	4.00	5.00	F.	0.21	10	3.68	0.92	17	3 720	5.91	691	339	47.2	386
3		8′ 0″	4.00	5.00	F.	0.21	10	3.68	0.92	17	3 470	4.05	691	316	47.2	363

TABLE 3.—(*Continued.*)

Slab No.	Reinforcement.	Span.	Thickness. Net.	Thickness. Gross.	End condition.	Percentage of steel.	Age, in weeks.	(*jd*)	(*kd*)	(*n*)	Maximum applied load, in pounds.	Maximum deflection, in inches.	Estimated weight, in pounds.	Equivalent load, in pounds per square foot.	Uniform dead load, in pounds per square foot.	Equivalent total uniform load, in pounds per square foot.
5-ft. Tw.																
1	¼ square twisted. Area, 0.178 in.	5' 0"	3.10	4.10	F.	0.26	4	2.79	0.96	26	3 500	0.41	336	509	36.5	546
2		5' 0"	3.45	4.20	F.	0.24	4	3.10	1.07	26	3 590	1.62	346	522	37.6	560
5-ft. Sq.																
1	¼ square bars. Area, 0.202 in.	5' 0"	3.27	4.27	F.	0.28	4	2.94	1.02	26	3 725	0.33	352	543	38.3	581
2		5' 0"	3.25	4.25	F.	0.28	4	2.93	1.01	26	3 825	0.48	351	556	38.2	594
5-R.																
1	¼ round bars. Area, 0.153 in.	5' 0"	3.20	4.20	F.	0.22	4	2.88	0.93	26	2 125	0.06	336	309	36.5	346
2		5' 0"	3.25	4.25	F.	0.21	4	2.94	0.94	26	1 575	0.04	351	229	38.2	267
5-PL																
1	No reinforcement	5' 0"	....	4.36	F.	....	4	....	....	..	1 500	0.019	360	219	39.3	258
2		5' 0"	....	4.25	F.	....	4	....	....	..	1 510	0.027	351	220	38.4	258
13-C-6																
1	Triangular wire mesh. Area, 0.173 in.	6' 0"	3.22	4.22	R.	0.25	4	2.96	0.81	16	11 182	0.85	437	1 357	39.6	1 397
2		6' 0"	3.22	4.22	R.	0.25	4	2.96	0.81	16	10 250	....	437	1 242	39.6	1 282
3		6' 0"	3.00	4.00	R.	0.26	4	2.76	0.75	16	13 500	0.73	414	1 636	37.6	1 674
4		6' 0"	3.37	4.37	R.	0.23	10	3.10	0.84	16	12 100	1.06	453	1 466	41.3	1 507
5		6' 0"	3.00	4.00	R.	0.26	10	2.76	0.75	16	12 295	1.21	414	1 490	37.6	1 528
6		6' 0"	3.10	4.10	R.	0.25	10	2.85	0.78	16	12 000	0.65	425	1 455	38.6	1 494
13-6																
1	" " " "	6' 0"	3.10	4.10	F.	0.25	5	2.85	0.78	16	3 900	....	425	473	38.6	512
2		6' 0"	3.10	4.10	F.	0.25	5	2.85	0.78	16	3 400	2.07	425	412	38.6	451
13-6																
1	Welded wire mesh. Area, 0.238 in.	6' 0"	3.27	4.27	F.	0.33	5	2.98	0.92	16	3 250	2.35	442	394	40.1	434
2		6' 0"	3.02	4.02	F.	0.36	5	2.75	0.85	16	3 900	3.00	416	473	37.8	511
3		6' 0"	3.07	4.07	F.	0.35	5	2.80	0.86	16	3 150	1.98	421	382	38.2	420
13-6																
1	Expanded metal. Area, 0.130 in.	6' 0"	3.05	4.05	F.	0.19	5	2.83	0.64	16	2 000	0.44	419	243	38.1	281
2		6' 0"	3.15	4.15	F.	0.18	5	2.93	0.66	16	2 100	0.63	430	254	39.1	293
3		6' 0"	3.07	4.07	F.	0.18	5	2.86	0.65	16	2 250	1.05	421	273	38.3	311

for the thrust produced by the test span, was varied by each individual tester. Consequently, no standard practice was developed, and the test results obtained were not comparable. Frequently, the test span

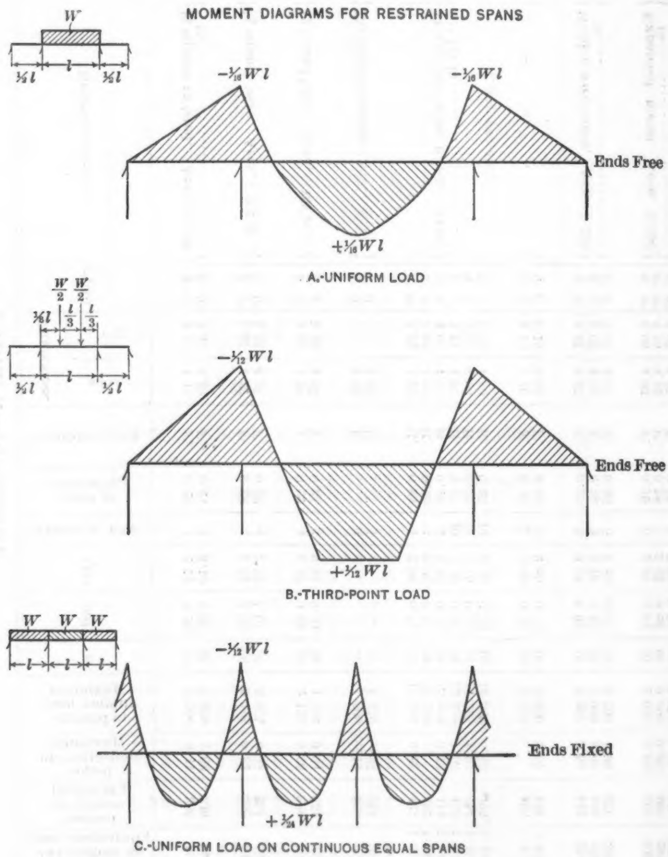


FIG. 20.

was rigidly held between two adjacent anchor-spans filled solidly with concrete, thus forming heavy haunches on both ends of the test span; and the reinforcement was securely anchored in these adjacent spans.

Observations of the tests led to the conclusion that, during the early stages of loading, the construction did not differ materially from the type of slab constructed continuously over two or more rigid supports. In planning the present series of tests, it was decided, therefore, to base the form of test specimen on the principles underlying the continuous type of beam. A comparison of the bending moments and shears for three types of loading are shown in Figs. 20 and 22 for the slab continuous over three spans, and in Fig. 21 for the simply supported slab. The factors for shears and moments in the former case were derived by the theorem of three moments, assuming full continuity of construction, fixed supports on the same

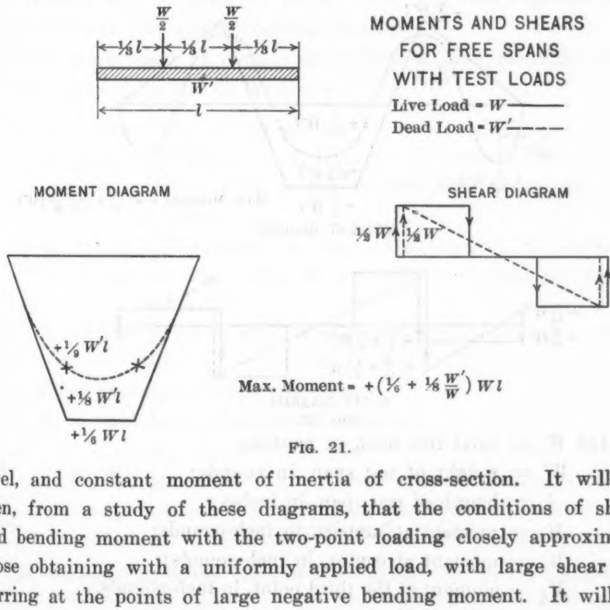


FIG. 21.

level, and constant moment of inertia of cross-section. It will be seen, from a study of these diagrams, that the conditions of shear and bending moment with the two-point loading closely approximate those obtaining with a uniformly applied load, with large shear occurring at the points of large negative bending moment. It will be observed, also, that the anchor-spans were in all cases assumed equal in length to one-half the test span, the purpose being to obtain the coefficients of bending moment ordinarily applied in the design of slabs continuous over two or more panels.

In all the tests on the simply supported and the restrained types of floors, the conditions of loading as outlined in Figs. 21 and 22

apply, respectively. The extreme ends of the specimens were in all cases considered to be freely supported, and the reinforcement was not secured at these points.

MOMENTS AND SHEARS FOR RESTRAINED SPANS WITH TEST LOADS

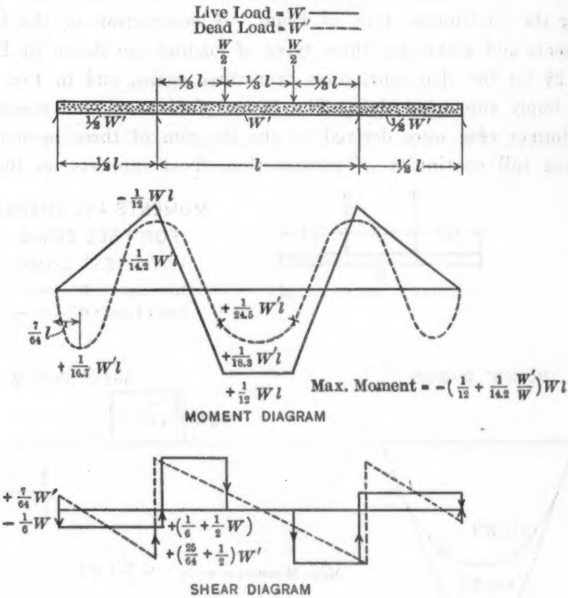


FIG. 22.

Let W = total live load, in pounds;

W' = weight of test span, in pounds;

l = length of test span, in inches;

M_s = moment at support, in inch-pounds;

M_c = moment at center, in inch-pounds;

and $M_{\frac{1}{3}}$ = moment at the third point, in inch-pounds.

Then, for the continuous or restrained construction,

$$M_s = +\left(\frac{1}{12} + \frac{1}{14.2} \times \frac{W'}{W}\right) W\,l \ldots\ldots\ldots\ldots (1)$$

$$M_c = +\left(\frac{1}{12} + \frac{1}{18.3} \times \frac{W'}{W}\right) W\,l \ldots\ldots\ldots\ldots (2)$$

$$M_{\frac{1}{3}} = +\left(\frac{1}{12} + \frac{1}{24.5} \times \frac{W'}{W}\right) W\,l \ldots\ldots\ldots\ldots (3)$$

and, for the simply supported or free spans,

$$M_c = + \left(\frac{1}{6} + \frac{1}{8} \times \frac{W'}{W}\right) W\, l \dots\dots\dots\dots (4)$$

$$M_{\frac{1}{3}} = + \left(\frac{1}{6} + \frac{1}{9} \times \frac{W'}{W}\right) W\, l \dots\dots\dots\dots (5)$$

It must be borne in mind that these formulas are to be applied only within the first stage of the test, as later described, which occurs within the region of working loads. With the third-point method of test, it is only necessary to be reasonably careful to maintain the load symmetrical with respect to the loading zones, as a 10% variation in one concentration as compared to the other will result in less than 4% variation in the applied bending moments.

The formula used in computing deflections is a modification of the form derived by Mr. G. A. Maney.*

Let y = deflection, in inches;
d = depth of slab to steel, from top of concrete, in inches;
e_c = unit deformation in the extreme fiber of the concrete, in inches;
e_s = unit deformation in steel reinforcement, in inches;
f_c = extreme fiber stress in concrete, in pounds per square inch;
f_s = unit stress in steel reinforcement, in pounds per square inch;
E_s = modulus of elasticity of steel;
E_c = modulus of elasticity of concrete;

and n = ratio $\frac{E_s}{E_c}$.

Then

$$y = \frac{k\, l^2}{d\, E_s} \times (n\, f_c + f_s) \dots\dots\dots\dots (6)$$

The constant, k, is the ratio of the quantity, k_1, in the expression for the deflection of a homogeneous beam, $k_1 \frac{W l^3}{E I}$ to the constant, k_2, in the expression for the maximum bending moments, $k_2\, W\, l$. The expressions for k_1 and k_2, with the loading under consideration, are as follows:

* *Proceedings*, Am. Soc. for Testing Materials, 1914.

For the continuous or restrained span, homogeneous beam,

$$y_1 = \left(\frac{1}{96} - \frac{1}{910} \times \frac{W'}{W}\right) \frac{W l^3}{E I} \text{..........(7)}$$

or

$$k_1 = \left(\frac{1}{96} - \frac{1}{910} \times \frac{W'}{W}\right) \text{..............(8)}$$

and, from Equation (2), $M_c = \left(\frac{1}{12} + \frac{1}{18.3} \times \frac{W'}{W}\right) W l$

or

$$k_2 = \left(\frac{1}{12} + \frac{1}{18.3} \times \frac{W'}{W}\right) \text{..............(9)}$$

whence,

$$k_R = \frac{\frac{1}{96} - \frac{1}{910} \times \frac{W'}{W}}{\frac{1}{12} + \frac{1}{18.3} \times \frac{W'}{W}} \text{......................(10)}$$

For the freely supported span, homogeneous beam,

$$y_2 = \left(\frac{1}{56.4} + \frac{1}{76.8} \times \frac{W'}{W}\right) \frac{W l^3}{E I} \text{........(11)}$$

or

$$k_1 = \left(\frac{1}{56.4} + \frac{1}{76.8} \times \frac{W'}{W}\right) \text{.............(12)}$$

and, from Equation (4),

$$M_c = + \left(\frac{1}{6} + \frac{1}{8} \times \frac{W'}{W}\right) W l$$

or

$$k_2 = \left(\frac{1}{6} + \frac{1}{8} \times \frac{W'}{W}\right) \text{.................(13)}$$

whence,

$$k_F = \frac{\frac{1}{56.4} + \frac{1}{76.8} \times \frac{W'}{W}}{\frac{1}{6} + \frac{1}{8} \times \frac{W'}{W}} \text{.................(14)}$$

(The derivation of all the formulas used in this paper will be found in Appendix V.)

The moment and shear diagrams for the special type of test slab are shown by Fig. 23. This form of specimen was tested in order to determine the comparative strength of an end panel not provided with an adjacent anchor-span to take up the thrust of the test span. This condition is found in end panels adjacent to the exterior walls of a building and in bays adjacent to elevators and stair-wells.

Analysis of Slab Action.

On Figs. 32 to 38 are shown the curves expressing the relation between the load and the measured compressive and tensile strains,

with the corresponding observed deflections. It must be borne in mind that the compressive strain was observed at a plane ½ in. down from the extreme fiber. To obtain the extreme fiber strain this measurement must be increased by the ratio of the distances from the neutral axis to the extreme fiber and to this point of measurement.

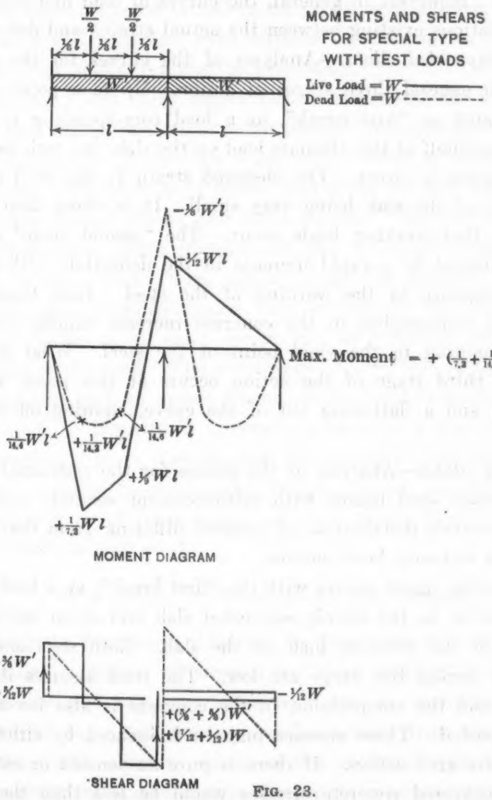


FIG. 23.

As noted on page 566, the tensile strain was measured on the exterior surface of the concrete at the plane of the steel reinforcement. It was to be expected, and subsequent results proved, that the true stretch of the steel could not be obtained in this manner, but the size of the reinforcing members and the thickness of this type of con-

struction prevent a more accurate treatment. Considering the shallowness of the slab and the small depth to the neutral axis, which was approximately 1 in., slight variations in the measurements, and the conversion of the observations into stresses, would lead to considerable error. Abnormalities are particularly to be expected in the steel stresses. However, in general, the curves of load and strain indicate the relations existing between the actual strains and deflections.

Simply Supported Slabs.—Analyses of the curves for the simple slabs show in general three stages of action. Up to a point on the curve designated as "first break", at a load corresponding to about one-third to one-half of the ultimate load on the slab, the ratio between load and stresses is direct. The observed strain in the steel is low, the deflection of the slab being very small. It is along this region of the curve that working loads occur. The "second stage" of the action is indicated by a rapid increase in the deflection, with a corresponding response in the working of the steel. Both tension in the steel and compression in the concrete increase rapidly up to a point corresponding to the yield point of the steel. What may be designated a third stage of the action occurs at this point, with a second break and a flattening out of the curve, running off rapidly to failure.

Restrained Slabs.—Analysis of the curves for the restrained slabs, built in between steel beams with reinforcement securely anchored, indicates a possible distribution of stresses differing from that to be expected from ordinary beam action.

The first stage again occurs with the "first break", at a load about twice as great as in the simply supported slab and about one-fourth to one-third of the ultimate load on the slab. Both steel and concrete stresses during this stage are low. The steel assumes its load very slowly, and the compression in the concrete is also lower than would be expected. These stresses may be influenced by either suspension or pure arch action. If there is pure suspension or catenary action, the measured concrete stresses would be less than the computed ones by the amount of this pure tension, and the steel stresses would be correspondingly large. If there is arch action, the steel stress would be low and the concrete compression would be distributed over a greater portion of the cross-section, giving possibly a lower unit intensity at the extreme fiber than would be indicated by the

ordinary theory of flexure. In the application of the ordinary theory of flexure, assumptions are made as to the relation between the concrete and steel stresses and the ultimate compressive value of concrete in flexure, about which there is little or no definite information. The flat slab held between rigid I-beam supports, with the resistance to lateral motion offered by adjoining panels, will develop considerable strength or end restraint, even without reinforcement. This is the so-called arch action referred to previously.

The second and third stages follow in general the action of the simple slab, except that the "curve breaks" for correspondingly reinforced slabs occur at absolute loads about twice as great as for the simple slab. The slab itself under load gives its first sign of failure in negative tension cracks over the haunches. Thereafter the adjoining anchor-spans deflect upward and the test span downward. Numerous tension cracks then appear along the bottom of the arch, developing most rapidly under one or both of the concentrations. Ultimate failure in all cases except two, as hereafter noted, occurred in tension in the steel. The percentage of steel was in all cases made purposely low, both to insure tension failure and because this type of construction in practice is generally used with a low percentage of reinforcement. It is possible that the concrete was stressed to its ultimate capacity preceding the failure in the reinforcement, but signs of crushing of the concrete were not visible to the eye until the tension break of the steel.

Conversion of Observations into Stresses.

During the first stage of action, in both types of construction, the straight-line variation of stresses has been assumed. The modulus of elasticity used has been taken from the stress-strain curves, Fig. 31, obtained from cylinders cast with concrete from the corresponding slabs. The absolute value of the modulus was taken as the average along the one-quarter to three-eighths points of the curve. It must be recognized that the modulus thus obtained is based on concrete under conditions of stress which do not represent those obtaining in the slab proper. As shown later, the modulus obtained from unreinforced slabs in flexure is considerably higher than that obtained from the cylinders.

The neutral axis was located and the factors given in Table 3 were derived on the assumptions usually made in applying the common theory of flexure for the composite material of concrete and steel. The ratio, n, was based on a modulus of elasticity of 30 000 000 lb. per sq. in. for steel. On Fig. 24 the symbols corresponding to the quantities given in the tables are indicated.

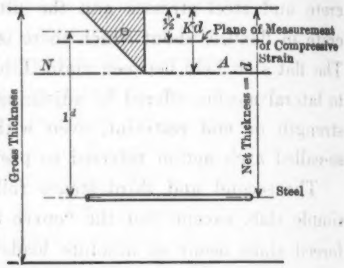


FIG. 24.

The tensile stress in the steel and the extreme fiber stress in compression are obtained in the following manner:

Let E_c = modulus of elasticity of the concrete, in pounds per square inch;

E_s = modulus of elasticity of the steel, in pounds per square inch;

n = ratio, $\frac{E_s}{E_c}$;

f_s = tension in steel, in pounds per square inch;

f_c = compression in extreme fiber, in pounds per square inch;

e_s = the measured unit tensile strain;

and e_c = the measured unit compressive strain.

Then
$$f_s = E_s \times e_s \quad \ldots\ldots\ldots\ldots (15)$$

and
$$f_c = E_s \times \frac{k\,d}{k\,d - \frac{1}{2}} \times e_c \quad \ldots\ldots\ldots\ldots (16)$$

Computation of Derived Bending Moment Factors.

With the stresses in concrete and steel known, in addition to the total load applied on any one span of known thickness, the corresponding bending moment factor to be applied to produce these stresses can readily be ascertained. In Table 15 this factor has been derived in four different ways, based on the following quantities:

1.—Measured compression at known load;
2.—Measured tension at known load;
3.—Measured compression and deflection at known load; and
4.—Ratio of imposed loads on simple and restrained slabs.

The stress in the steel can be determined in two ways: First, from the observed deformation at the plane of the steel (Equation (15)); and second, from the observed deformation in the concrete and the observed deflection of the slab (Equations (6) and (16)).

Let w = load, in pounds per square foot at a point below the first break;
f_c = corresponding compressive stress measured;
f_s = corresponding steel stress measured;
y_1 = observed deflection, in inches;
f'_s = computed tension based on f_c and y_1;
k = ratio of depth of neutral axis to depth of steel;
j = ratio of effective depth to depth of steel;
L = length of span, in feet;
A_s = area of steel, in square inches;
M_r = resisting moment of section;
M_i = implied bending moment, $12 \frac{1}{c} w L^2 \times \frac{22}{12}$;

and $\frac{1}{c}$ = bending moment coefficient.

The width of slab is in all cases 22 in.

Method 1.—

$$M_r = 22 \times k\, d \times \frac{1}{2} f_c \times j\, d = 11\, f_c\, k\, j\, d^2 \qquad (17)$$

$$M_i = \frac{1}{c} \times w \times L \times \frac{22}{12} \times L \times 12 = \frac{22}{c} w\, L^2 \qquad (18)$$

And, from Equations (17) and (18),

$$c = \frac{2\, w\, L^2}{f_c\, k\, j\, d^2} \qquad (19)$$

Method 2.—

$$M_r = A_s \times f_s \times j\, d \qquad (20)$$

$$M_i = \frac{22}{c} w\, L^2 \qquad (21)$$

And, from Equations (20) and (21),

$$c = \frac{22\, w\, L^2}{A_s\, f_s\, j\, d} \qquad (22)$$

Method 3.—Assuming that the deformation measured in the exterior surface of the concrete in the plane of the steel reinforcement is

incorrect, and that the compressive deformation is more accurate, although it must be borne in mind that all such strain measurements are subject to various errors, then, from Equation (6), we have

$$y = \frac{k\,l^2}{d\,E_s}(n\,f_c + f_s').$$

Substituting for y the known deflection, and making also the proper substitution for the quantities, k, n, f_c, and E_s, the equation can be solved for f_s', the stress in the steel reinforcement, and from Equation (22), we have

$$c = \frac{22\,w\,L^2}{A_s\,f_s'\,j\,d} \qquad \text{.......................(23)}$$

Method 4.—The fourth method of determining the implied bending moment coefficient is based on a comparison of the ultimate loads supported by the simple and restrained types of specimens. These specimens were cast in groups of corresponding specifications, with both end conditions.

Let W_1 = equivalent uniform load, in pounds per square foot, supported by the simple slab;
and W_2 = equivalent uniform load, in pounds per square foot, supported by the restrained slab.

Then, since the bending moment coefficient for a simple slab loaded uniformly is $\frac{1}{8}$, the denominator of the coefficient for the restrained slab will be

$$c = 8\,\frac{W_2}{W_1} \qquad \text{.......................(24)}$$

These values for the coefficient have been computed in Table 15, Appendix V, as well as the corresponding stresses computed by the ordinary straight-line theory of flexure, and the observed stresses in concrete and steel for a point just below the first break in the deformation curves of the 6 and 7-ft. free and restrained slabs.

It must again be cautioned that the attempt to measure fiber deformations in the surface of the concrete is at best only an approximate operation. In the case of thin slabs, such as those with which the present investigation is concerned, the depth to the neutral axis is so small that a slight variation in the location of the contact points

for measurement may produce a very large error. Moreover, the selection of the proper modulus of elasticity is an indeterminate feature. The method of determining safe loads and moment coefficients by the absolute supporting power of the slabs, as indicated by Method 4, is not subject to such errors, and presumably leads to approximately accurate results.

Modulus of Elasticity of Unreinforced Slabs.

Two concrete slabs, 22 in. wide and 4 in. thick, of the same mix as the 5-ft. reinforced specimens, were tested on a simple span, and the deflections of the slab were accurately measured. Then, from the common theory of flexure, using Equation (12), we have

$$y = \left(\frac{1}{56.4} + \frac{1}{76.8} \times \frac{W'}{W}\right)\frac{W\,l^3}{E\,I}.$$

Substituting the value of y, and W, W', e, and I, the equation can be solved for E.

The values of the coefficient of elasticity thus obtained have been given in Table 16. It will be noted that these values are higher than those obtained for the cylinders in compression. Inasmuch as the tensile value of concrete is ordinarily neglected in the design of reinforced concrete beams and slabs in flexure, it is customary to assume a smaller value of the coefficient of elasticity to compensate for this neglect, which is equivalent to assuming a larger ratio of E_s to E_c, thus lowering the apparent location of the neutral axis in cases of small percentages of steel or over-design in concrete. In the conversion of strains to stresses in this investigation, the ratio, n, has been obtained from the actual coefficients of elasticity as determined from the cylinder tests, as this in itself is possibly lower in value than would be obtained from concrete under the conditions of stress existing in the slab.

Derivation of Empirical Formulas.

In the ordinary practice of cinder concrete floor construction between steel beams, as applied in New York City, the safe loads to be supported by the construction have been determined by load tests of various characters, as described in the early part of this paper. During the past two years, it has been suggested, by a joint committee of architects and engineers, that the results of the more satisfactory of these tests be combined in a table from which the safe load

for varying conditions of span and reinforcement can readily be obtained. Such a table was prepared and has been embodied in two of the drafts of a proposed code for New York City.

A study of the results of this investigation would indicate that this procedure is rather drastic, and that the working loads thus obtained are excessive in some cases and discriminating in others. An attempt was made, therefore, to develop empirical formulas which would check the theoretical design of floors of this type when based on the coefficients developed by the investigation.

Let $w_1 = \frac{1}{8}$ of the equivalent ultimate uniform load on the slab at failure, in pounds per square foot;

and $w_2 =$ the dead load or weight of the slab, in pounds per square foot.

Then the safe total load was assumed as $w_1 + w_2$. These safe loads are given in Table 4. A study of the influence of variations in area of steel reinforcement, thickness of slab, length of span, and conditions of end restraint was made with a view of determining the relative weights of these variables in fixing the safe load.

Let $a =$ area of steel, in square inches;
$l =$ length of span, in inches;
$d =$ depth of steel from extreme upper fiber, in inches;
and $W_t =$ total safe load, in pounds per square foot.

It was found that, for the restrained slabs,

W_t is proportional to $a^{0.84}$ ....(1)

W_t is proportional to $d^{0.99}$ ....(2)

W_t is proportional to $\frac{1}{l^{2.5}}$ ....(3)

The assumption was then made, therefore, that

$$W_t = K_1 \frac{d\,a}{l^2} \qquad (25)$$

This equation has the general form of the equation for the load on a suspension cable, expressed in terms of the horizontal tension in the lowest point of the cable and the sag or depth, that is

$$H = \frac{\frac{1}{8}\,w\,l^2}{d} = a\,s, \text{ or } w = 8\,S\,\frac{d\,a}{l^2}.$$

Substituting the corresponding values for W_t obtained from the tests in Equation (25), the following values were obtained for K_1:

*Official, 7 ft. 6 in. 4 300 000
*Official, 8 ft. 0 in. 2 800 000
C — 7 ft. 1 in. 3 600 000
C — 6 ft. 0 in.—1914. . . . . 3 400 000
C — 6 ft. 0 in.—1913. . . . . 3 940 000
C — 8 ft. 0 in.—1913. . . . . 3 150 000

From the foregoing the average value of K is approximately 3 500 000, or, for the restrained slabs,

$$W_t, \text{ in pounds per square foot, } = 3\,500\,000\,\frac{d\,a}{l^2} \quad \ldots\ldots (26)$$

In the case of plain bars hooked over the steel beams, without lateral reinforcement, Equation (26) does not apply. A study of all the available data would indicate that the following expression meets the conditions of that type of reinforcement:

$$W_t, \text{ in pounds per square foot, } = 2\,500\,000\,\frac{d\,a}{l^2}.$$

In the same manner, the safe total loads for the simple slabs were found, and the weight of each factor was determined. The following proportions were developed:

W_t is proportional to $a^{0.75}$ . . . . (1)
W_t is proportional to $d^{1.3}$ . . . . (2)
W_t is proportional to $\frac{1}{l^{1.3}}$ . . . . (3)

It was assumed, therefore, that

$$W = K_2\,\frac{d^2\,a}{l^2} \quad \ldots\ldots (27)$$

With this formula as a basis, the solution gave the following values of K_2:

7 ft. 1 in.—1914. . . . . 600 000
6 ft. 0 in.—1914. . . . . 565 000
A — 4 in. — 5 ft. 0 in.—1914. . . . . 425 000
A — 3 in. — 5 ft. 0 in.—1914. . . . . 540 000
Ex. 5 ft. 0 in.—1914. . . . . 840 000

* These data were obtained from the records of the Bureau of Buildings, from tests made under conditions practically identical with those of the investigation.

Tw.		5 ft. 0 in.—1914	415 000
Sq.		5 ft. 0 in.—1914	430 000*
Rd.		5 ft. 0 in.—1914	320 000*
A		6 ft. 0 in.—1913	555 000
Cl.		6 ft. 0 in.—1913	390 000
Ex.		6 ft. 0 in.—1913	550 000
A	4 in.	8 ft. 0 in.—1913	710 000
A	5 in.	8 ft. 0 in.—1913	520 000

The average value of the foregoing is approximately 550 000, whence, for the simple slabs unrestrained by steel beams and without anchored reinforcement,

$$W_t, \text{ in pounds per square foot, } = 550\,000 \frac{d^2 a}{l^2} \quad \text{.........(28)}$$

Bond Failure.—Four of the simple slabs of 5 ft. 0 in. spans, reinforced respectively with ¼-in. square and ¼-in. round rods spaced 8 in. apart on centers, failed by slipping of the bond of concrete and steel.

Let U = bond, in pounds per square inch;

ΣO = total circumference of the rods, in inches;

V = end reaction, in pounds.

Then
$$U = \frac{V}{\Sigma O j d} \quad \text{.........................(29)}$$

Computing the bond stress at failure by Equation (29), it was found that, at failure, U was 184 lb. per sq. in. for round rods and 255 lb. per sq. in. for square rods. On the other hand, the twisted square reinforcement which failed in tension was subject to a bond stress of 250 lb. per sq. in. Although these tests are not conclusive enough, the general indication is that a safe bond stress of about 40 lb. per sq. in. should not be exceeded in the case of unanchored or loose reinforcement.

In Table 4 have been compiled the safe loads as determined by test and the loads obtained from Equations (26) and (28). From a study of the typical cinder concrete gathered from about the city (Table 1), and the concrete used in the test slabs (Table 8), the following working units were selected as being best suited for the basis of design:

Extreme fiber stress in concrete.... 300 lb. per sq. in.

Tensile stress in steel............16 000 " " " "

* Bond failures, omit in averages.

TABLE 4.—SAFE LOADS, BASED ON TEST AND EMPIRICAL FORMULAS.

Restrained Type, $w = 3\ 500\ 000 \frac{d\ a}{l^2}$. Simple Type, $w = 550\ 000 \frac{d^2\ a}{l^2}$.

w = pounds per square foot, total load ; a = area of steel per foot of slab, in square inches ;
d = depth to steel, in inches ; l = length of span, in inches.

Type.	Safe Load. One-eighth of test load plus dead load.	Safe Load. Empirical formula.	Ratio of formula load to ultimate load.	Coefficient, $\frac{1}{c}$, Based on: Ratio, n = 30 and f_1 = 300 and Load, Column (2). Concrete.	Ratio, n = 30 and f_1 = 300 and Load, Column (2). Steel.	Ratio, n = 30 and f_1 = 300 and Load, Column (3). Concrete.	Ratio, n = 30 and f_1 = 300 and Load, Column (3). Steel.	Ratio, n, Determined by Test and Load, Column (3). Concrete.	Ratio, n, Determined by Test and Load, Column (3). Steel.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
C–7 ft. 1 in.	140	137	6.4	$\frac{1}{23.6}$	$\frac{1}{27.4}$	$\frac{1}{23.4}$	$\frac{1}{27.3}$	$\frac{1}{16.8}$	$\frac{1}{19.5}$
C–6 ft. 0 in., 1914	185	182	6.8	$\frac{1}{23.4}$	$\frac{1}{27.1}$	$\frac{1}{23.1}$	$\frac{1}{26.7}$	$\frac{1}{18.2}$	$\frac{1}{19.2}$
C–6 ft. 0 in., 1913	216	182	8.0	$\frac{1}{26.4}$	$\frac{1}{31.4}$	$\frac{1}{22}$	$\frac{1}{26.4}$	$\frac{1}{18.2}$	$\frac{1}{19.2}$
C–8 ft. 0 in., 1913	97	107	4.9	$\frac{1}{22.2}$	$\frac{1}{25.7}$	$\frac{1}{24.3}$	$\frac{1}{28.4}$	$\frac{1}{21}$	$\frac{1}{20}$
Official, 7 ft. 6 in.	274	225	7.7	$\frac{1}{17.5}$	$\frac{1}{28.0}$	$\frac{1}{15.1}$	$\frac{1}{23.0}$	$\frac{1}{11.2}$†	$\frac{1}{19.8}$
Official, 8 ft. 0 in.	172	214	5.3	$\frac{1}{32.0}$	$\frac{1}{25.5}$	$\frac{1}{39.8}$	$\frac{1}{31.6}$	$\frac{1}{28}$	$\frac{1}{21.2}$
Triangular, 7 ft. 1 in., 1914	70	65	5.0	$\frac{1}{10.0}$	$\frac{1}{11.8}$	$\frac{1}{9.8}$	$\frac{1}{11.7}$	$\frac{1}{7.2}$	$\frac{1}{8.6}$
Triangular, 6 ft. 0 in., 1914	92	90	5.5	$\frac{1}{11.1}$	$\frac{1}{13.3}$	$\frac{1}{10.9}$	$\frac{1}{13.1}$	$\frac{1}{8.2}$	$\frac{1}{9.1}$
Triangular, 4 in., 8 ft. 0 in., 1913	65	51	5.4	$\frac{1}{14.9}$	$\frac{1}{17.3}$	$\frac{1}{11.6}$	$\frac{1}{13.6}$	$\frac{1}{9.9}$	$\frac{1}{9.4}$
Triangular, 5 in., 8 ft. 0 in., 1913	85	90	4.1	$\frac{1}{12.0}$	$\frac{1}{16.3}$	$\frac{1}{12.6}$	$\frac{1}{17.3}$	$\frac{1}{11.3}$	$\frac{1}{12.5}$
Welded, 6 ft. 0 in., 1913	88	124	3.7	$\frac{1}{9.8}$	$\frac{1}{9.6}$	$\frac{1}{13.8}$	$\frac{1}{13.6}$	$\frac{1}{12.5}$	$\frac{1}{9}$
Exp., 6 ft. 0 in., 1913	68	97	3.05	$\frac{1}{9.7}$	$\frac{1}{14.7}$	$\frac{1}{13.9}$	$\frac{1}{20.3}$	$\frac{1}{12.5}$	$\frac{1}{12.9}$
Triangular, 6 ft. 0 in., 1913	90	90	5.3	$\frac{1}{11.3}$	$\frac{1}{13.5}$	$\frac{1}{11.3}$	$\frac{1}{13.5}$	$\frac{1}{9.7}$	$\frac{1}{9.1}$
4 in., 5 ft. 0 in., 1914.	100	129	4.2	$\frac{1}{8.2}$	$\frac{1}{9.5}$	$\frac{1}{10.5}$	$\frac{1}{12.3}$	$\frac{1}{7.4}$	$\frac{1}{8.9}$
3 in., 5 ft. 0 in., 1914.	56	57	4.5	$\frac{1}{9.1}$	$\frac{1}{8.5}$	$\frac{1}{9.1}$	$\frac{1}{8.5}$	$\frac{1}{6.7}$	$\frac{1}{6.1}$
Exp., 5 ft. 0 in., 1914.	88	98	4.6	$\frac{1}{6.7}$	$\frac{1}{10.6}$	$\frac{1}{7.4}$	$\frac{1}{11.7}$	$\frac{1}{5.4}$	$\frac{1}{7.7}$
Tw., 5 ft. 0 in., 1914	100	133	4.2	$\frac{1}{7.7}$	$\frac{1}{9.2}$	$\frac{1}{10.3}$	$\frac{1}{12.2}$	$\frac{1}{8.7}$	$\frac{1}{7.5}$
Sq., 5 ft. 0 in., 1914*.	105	150	3.9	$\frac{1}{7.9}$	$\frac{1}{8.7}$	$\frac{1}{11.3}$	$\frac{1}{12.4}$	$\frac{1}{8.4}$	$\frac{1}{8.7}$
Rd., 5 ft. 0 in., 1914*.	70	115	2.7	$\frac{1}{6}$	$\frac{1}{8.2}$	$\frac{1}{9.8}$	$\frac{1}{13.4}$	$\frac{1}{7.1}$	$\frac{1}{8.8}$

* Failure in bond.

† This slab had 1 : 2 cement finish 1 in. thick ; if an extreme fiber stress of 600 lb. per sq. in. is assumed, which makes $\frac{1}{c} = \frac{1}{24}$.

With these factors as a basis of computation, using the value of the ratio, n, as determined by test on the slab concrete in Columns 9 and 10, and a ratio of 30 in Columns 5, 6, 7, and 8, of Table 4, the corresponding factor for implied bending moments was derived, as shown in that table. The factor in the bending moment formula, $\frac{1}{c} W l$, for the simple slabs, was found to be approximately $\frac{1}{8}$, as expected, and for the restrained slabs approximately $\frac{1}{20}$. (See Columns 9 and 10 of Table 4.)

No special care was taken in preparing the concrete for these tests, nor was the material specially selected. It is to be presumed, therefore, that the results are indicative of what may be expected from cinder concrete placed with ordinary care and under proper supervision. With the foregoing loads considered as safe, the average factor of safety on the ultimate load basis is approximately 5.

Acknowledgments.

In addition to those already mentioned, the writers wish to make the following acknowledgments: The tests were conducted by the Department of Civil Engineering, Columbia University, partly with the co-operation of the Bureau of Buildings, Borough of Manhattan, and were made possible through the action of William H. Burr, M. Am. Soc. C. E., who made available the William Richmond Peters, Jr., Fund for Engineering Research. The Bureau was represented in the field by Mr. K. M. Boorman, Engineer-Inspector. The actual tests were run by Messrs. F. Miller and J. P. Maloy, Assistants in the Department of Civil Engineering, Columbia University, and, in addition, in 1913, by E. H. Kirchgraber, Jun. Am. Soc. C. E., and Mr. G. Lommel, and, in 1914, by Messrs. S. Quattlebaum, G. Richardson, and H. C. Stevens, all senior students in Civil Engineering at Columbia University.

Conclusions and Recommendations.

From general experience with anthracite clinkers, and knowledge gained from the present investigation, the following conclusions may be drawn, and on these the writers have based their recommendations.

As to Fire Resistance.—

1.—Cinder concrete, even though cast in a lean mixture and of low-grade material, is an extremely effective fire-resisting product.

2.—Steel, especially if of small cross-section and high carbon content, placed within 1 in. of the exposed surface is apt to be annealed to a marked extent, with consequent loss in strength, when the embedding concrete is subjected to fire, under conditions obtaining in conflagrations. This observation, however, might easily be made with reference to concrete mixed from any common aggregate.

3.—Under ordinary conditions, the pitting which occurs to a depth probably not greater than 1 in., even in extreme cases, and is caused by the combustion of particles of exposed coal and by the effect of de-hydration, may be readily repaired by plastering directly on the damaged surface, without the necessity of lathing or other means of support.

As to Corrosive Effects.—

1.—Anthracite cinder concrete (1:2:5), well mixed, cast in a viscous to wet consistency, constantly stirred and mixed during placement, in such a manner as to coat the reinforcement thoroughly with mortar, will not cause the corrosion of embedded steel.

Table 5 shows the chemical constituents of the cinders, as determined by analysis. The first four samples represent the cinders used in the concretes reported as typical New York material. The last two samples represent the aggregate used in the test slab concrete.

TABLE 5.—CHEMICAL CONSTITUENTS OF CINDERS.

Samples.	Percentage of unburned coal determined by carbon loss on ignition.	Percentage of sulphides.	Percentage of sulphuric anhydride. (SO_3)
A	16.50	0.073	0.21
B_1	28.10	0.091	0.14
B_2	25.75	0.084	0.08
C	26.41	0.107	0.07
Slabs, 1913	15.96	0.063	0.09
Slabs, 1914	10.84	0.056	0.03

It will be observed that the quantity of sulphur is in general small and increases with the unburned coal content. The sulphur existing as sulphides is probably inert. The sulphuric anhydride, which is present in the cinders as sulphates, may be an agent of corrosion, in the following manner. If in the form of iron sulphate, chemical action may result in the presence of the cement and moisture, resulting in the formation of calcium sulphate and iron oxide. Professor

Norton has shown that incipient corrosion has occurred where iron oxide is in contact with the steel. The only safeguard is to insure the presence of a protective coating of cement mortar on the steel reinforcement, by the use of a mixture of a viscous or wet consistency.

As to the Characteristics of the Typical New York Product.—

1.—The variableness of New York cinder concrete is marked. Other conditions being identical, a difference of 150% in compressive strength might be expected to be shown by two series of specimens, due alone to the quality of the cinders.

2.—Hand-mixed cinder concrete handled in the ordinary manner is inferior to the machine-mixed product.

3.—The average weight of reinforced cinder concrete, based on actual investigation, was found to be 108 lb. per cu. ft. It is recommended that this be the minimum value assumed in design.

4.—The crushing strength of standard cylinders of good cinder concrete as used in New York is between 800 and 1 000 lb. per sq. in. at 28 days. Poor material will give results as low as 400 lb. per sq. in.

5.—The crushing strength increases for a period of about 6 months, after which it remains practically constant for a period of a year.

6.—The modulus of elasticity, *E*, under working loads, determined from standard cylinders, is about 1 200 000 lb. per sq. in. for average concrete at 28 days. It varies only slightly with age.

As to Proportions and Mixing in General.—

1.—The assumption that screened cinders will produce better concrete was investigated, with the result that 1 : 2 : 5 concrete, made with cinders from which all material less than ¼ in. diameter had been removed, showed a decrease in compressive strength of more than 25 per cent. (See Appendix IV.)

2.—Cinder concrete should be mechanically-mixed, for not less than 2 min., not leaner than 1 : 2 : 5, giving a minimum compressive strength, with standard cylinders at 28 days, of 800 lb. per sq. in., and be in a wet to viscous consistency when cast. It should be made from hard, well-burned, vitreous clinkers, free from fine, powdery ash. It has been found from actual test that unburned coal, existing to the extent of from 15 to 25%, is not necessarily detrimental to the strength or fire-resisting value of such concrete.

3.—Large irregular clinkers, often 2, 3, or 4 in. or even larger, in diameter, should be thoroughly broken up, by shovels or otherwise, before mixing, as they are sources of weakness in the finished product, if allowed to remain intact. This is due, not only to the inherent weakness of the aggregate itself, but also to the difficulty with which the mortar may reach all parts of its porous surface, a condition necessary for maximum density.

As to Future Regulation.—

1.—The safe load of cinder concrete floor fillings should be determined by any one of the following methods:

A.—Design.

a.—Common Theory of Flexure.

For slabs restrained between steel beams, with reinforcement thoroughly anchored, such as continuous mesh with proper cross-wires:

Bending moment coefficient.....	$\frac{1}{20}$
Extreme fiber stress in concrete..	300 lb. per sq. in.
Steel stress	16 000 " " " "
Ratio, n	30

For slabs simply supported with non-continuous reinforcement:

Bending moment coefficient....	$\frac{1}{8}$
Extreme fiber stress............	300 lb. per sq. in.
Steel stress	16 000 " " "
Ratio, n	30
Bond stress	40 lb. per sq. in.

Note that the safe bond stress of 40 lb. per sq. in. should be applied to loose bar reinforcement. In the case of mesh, the cross-wires give sufficient mechanical bond.

b.—Empirical Formula.

Restrained slabs, total safe load per square foot $= 3\,500\,000 \frac{d\,a}{l^2}$.

To be applied to continuous mesh with proper cross-wires.

Restrained slabs, total safe load $= 2\,500\,000 \frac{d\,a}{l^2}$. To be applied to longitudinal bars hooked over the beams.

Simple slabs, total safe load $= 550\,000 \frac{d^2\,a}{l^2}$.

In the application of the last formula, investigation should be made of the bond stress when loose bars are used.

Where $d =$ depth of steel, in inches;

$a =$ area of longitudinal steel per foot of width of slab, in square inches;

$l =$ length of span, in inches;

$w =$ load, in pounds per square foot, including the weight of slab.

The span should be limited to 8 ft. in the application of the special formulas given herein, and the minimum total thickness to one-eighteenth of the span length. It should also be noted that the empirical formulas are based on tests of slabs reinforced with steel varying from $\frac{1}{4}$ to $\frac{3}{8}$ per cent. For the stresses assumed in this analysis, the critical percentage of steel is about 0.32. (Table 17.)

For end-span conditions, the safe load, as determined by the formulas, should be reduced to three-fifths of the foregoing recommended values for the restrained type; or the strength of the construction should be increased proportionately.

B.—Load Test.

The load test should be conducted on a section of floor system not less than 4 ft. wide and with a span between steel beams equal to the span for which approval is desired. This specimen should be constructed as in actual practice, with material of the same quality as that incorporated in the specimen previously subjected to the standard fire, load, and water tests, and which is thereafter to be used in the system. In all cases the specimen should consist of a test span of normal dimensions and two anchor-spans, each of one-half the length of the test span. These anchors should conform to the construction of the test bay in every detail except in length.

In the case of reinforcement manufactured in sheets, which is to be put in with laps between sheets, the lap must come within the middle third of the test span.

The specimen should be tested either by stacking pig iron, bags of sand, or any other available material of sufficient specific gravity, on a platform supported on timbers placed so that the test load will be applied at two points, or loading areas, at the one-third points of the test span. Each loading area should consist of a zone of the

slab surface not exceeding 1 ft. in width and of a length conforming to the width of the test slab. A loose sand bed may be provided, not more than 1 in. thick, at each loading area or zone. Or the specimen should be loaded similarly in a testing machine, by a calibrated hydraulic jack, or by other mechanical means. The construction should be tested at the age of 1 month.

It is recommended that approval be granted for a live load of not more than one-eighth of the equivalent uniform imposed load necessary to cause the failure of the slab. A factor of safety of 8, on the live load basis, is equivalent to approximately 6 on the total load in the ordinary case of modern practice.

APPENDIX I

Typical New York Material.
Curves and Additional Data.

TABLE 6.—Mechanical Analysis of Sands and Cinders Used in Casting Typical New York Product.

Zone.	Mix.	Material.	Percentage Passing Sieve.									
			1¼-in.	1-in.	¾-in.	½-in.	¼-in.	No. 10.	No. 30.	No. 40.	No. 50.	No. 100.
A	1 : 2 : 5	Sand........	......	......	......	100.0	98.8	88.9	58.5	39.5	29.3	7.3
		Cinders........	97.8	97.2	93.4	85.9	63.4	38.8	24.4	22.8	21.6	13.7
B_1	1 : 1 : 5	Sand........	......	......	......	100.0	96.4	79.0	62.9	41.4	30.3	7.6
		Cinders........	100.	98.2	93.4	76.1	34.1	11.1	5.0	3.7	3.5	1.4
B_2	1 : 2 : 5	Sand........	......	......	......	100.0	98.3	75.9	56.3	33.4	25.3	6.5
		Cinders........	94.4	......	80.3	57.3	24.3	11.7	9.3	6.7	6.2	3.6
C	1 : 2 : 5	Sand........	......	......	......	100.0	99.2	80.8	57.0	38.3	25.8	4.2
		Cinders........	98.1	96.3	88.3	64.5	25.4	11.1	3.0	1.5	1.3	0.5

TABLE 7.—Tests on Cements Used in Casting Typical New York Product.

Zone.	Brand.	Specific gravity.	Percentage Finer than Sieves:		Percentage of water.	Tensile Strength:				
			No. 100.	No. 200.		24 hours.	7 days.	28 days.	56 days.	1 year.
A	Alsen..........	3.180	94.5	82.2	22½	308	566	695	713	725
B_1	Dragon.......	3.175	96.7	82.9	23	328	612	566	670	653
B_2	Vulcanite....	3.190	96.4	84.7	23	209	...	648	702	653
C	Atlas.........	3.150	97.7	81.2	22½	301	476	602	638	622

TABLE 8.—Typical Cylinder Test.

C —1 Month—No. 4.

Diameter = 8.07 in. Weight = 50 lb. 4 oz.

Height = 16.00 in. Area = 51.1 sq. in.

Total load, in pounds.	Unit stress, in pounds per square inch.	Unit deformation, in inches.
500	10	0.000000
2 500	49	0.000030
4 500	88	0.000075
6 500	127	0.000117
8 500	166	0.000150
10 500	205	0.000183
12 500	245	0.000233
14 500	284	0.000333
17 500	342	0.000375
20 500	401	0.000466
23 500	460	0.000573
26 500	519	0.000725
29 500	578	0.000800
32 500	636	0.000960
35 500	695	0.001185
41 100	804 Ultimate strength, in pounds per square inch.	

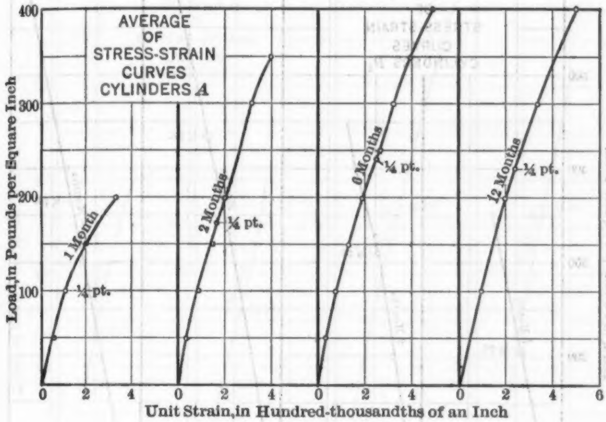

AVERAGE
OF
STRESS-STRAIN
CURVES
CYLINDERS A
1 Month
2 Months
6 Months
12 Months
¼ pt.
Load, in Pounds per Square Inch
Unit Strain, in Hundred-thousandths of an Inch

FIG. 25.

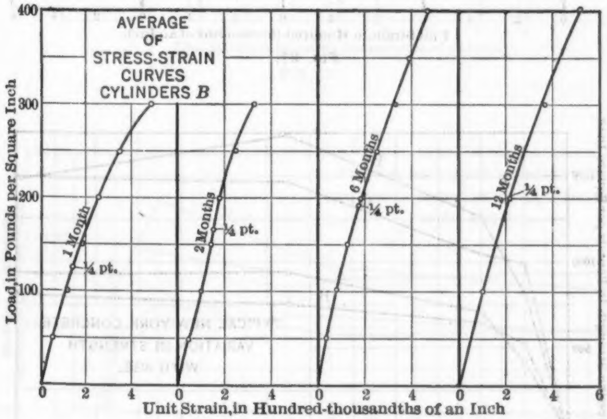

AVERAGE
OF
STRESS-STRAIN
CURVES
CYLINDERS B
1 Month
2 Months
6 Months
12 Months
¼ pt.
Load, in Pounds per Square Inch
Unit Strain, in Hundred-thousandths of an Inch

FIG. 26.

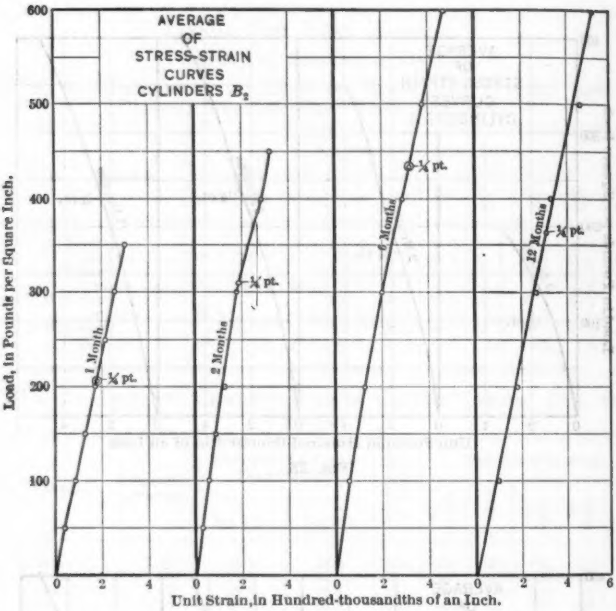

AVERAGE
OF
STRESS-STRAIN
CURVES
CYLINDERS B_2
Load, in Pounds per Square Inch.
1 Month
2 Months
6 Months
12 Months
¼ pt.
Unit Strain, in Hundred-thousandths of an Inch.

FIG. 27.

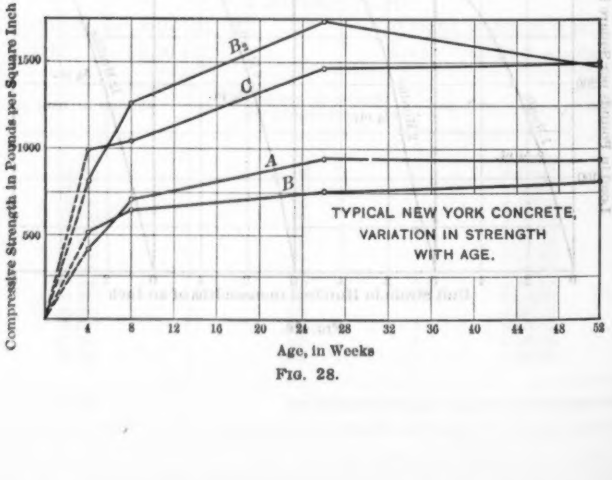

Compressive Strength in Pounds per Square Inch
B_2
C
A
B
TYPICAL NEW YORK CONCRETE,
VARIATION IN STRENGTH
WITH AGE.
Age, in Weeks

FIG. 28.

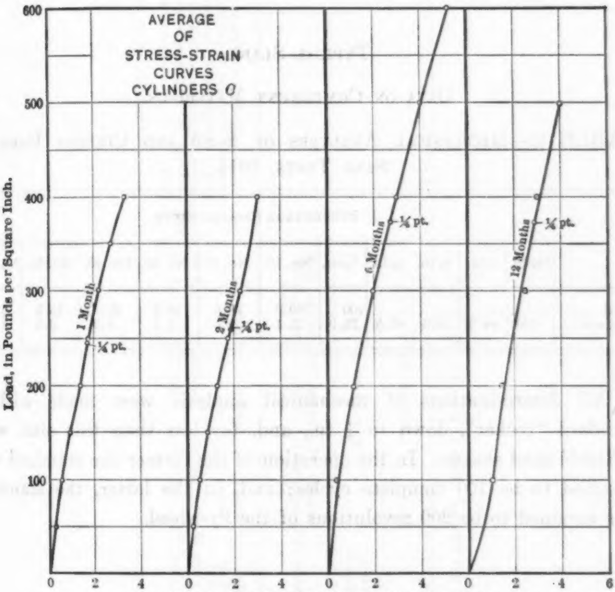

AVERAGE
OF
STRESS-STRAIN
CURVES
CYLINDERS C
Load, in Pounds per Square Inch.
Unit Strain, in Hundred-thousandths of an Inch.
1 Month
2 Months
6 Months
12 Months
¼ pt.
600
500
400
300
200
100
0
2
4
6

FIG. 29.

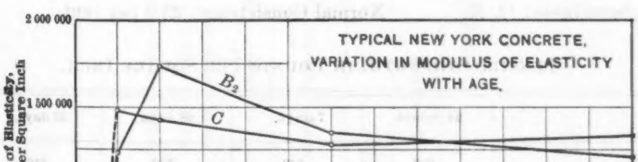

TYPICAL NEW YORK CONCRETE,
VARIATION IN MODULUS OF ELASTICITY
WITH AGE.
Modulus of Elasticity, in Pounds per Square Inch
Age, in Weeks
2 000 000
1 500 000
1 000 000
B_2
C
A
B
4
8
12
16
20
24
28
32
36
40
44
48
52

FIG. 30.

APPENDIX II

Typical Slabs.

Data on Component Materials.

TABLE 9.—Mechanical Analyses of Sand and Cinders Used in Slab Tests, 1914.

	Percentage Passing Sieve.										
	1¼-in.	1-in.	¾-in.	½-in.	¼-in.	No. 10.	No. 16.	No. 30.	No. 40.	No. 50.	No. 100.
Sand	......	......	......	......	100	89.0	78.8	48.3	29.7	15.5	2.25
Cinders	100	94.3	90.3	80.5	73.9	15.1	9.5	5.1	4.2	3.4	2.3

All determinations of mechanical analysis were made with a standard "rocker", down to ¼ in., and, for less than this size, with a Riehlé sand shaker. In the operation of the former the standard was assumed to be 100 complete cycles; and, in the latter, the standard was assumed to be 200 revolutions of the fly-wheel.

TABLE 10.—Tests on Cement Used in Slab Tests, 1914.

Brand: Dragon — Percentage Passing No. 100 Sieve: 96.3

Specific Gravity: 3.13 — " " " 200 " 83.4

Soundness: O. K. — Normal Consistency, 25.0 per cent.

Tensile Strength, in Pounds per Square Inch.

	24 hours.	7 days.	28 days.	56 days.
Neat	202	643	702	616
Sand	....	199	328	352

Compressive Strength, in Pounds Per Square Inch, on 2-In. Cubes.

Neat	1 269	5 606	6 132	6 952
Sand	....	1 468	1 445	1 521

TABLE 11.—Tensile Tests on Reinforcement.

	1913 Series.			1914 Series.					
	Cold-drawn wire.			Cold-drawn wire, triangular mesh.		Rods.			Expanded metal.
	6-ft. spans, triangular mesh.	6-ft. spans, welded mesh.	8-ft. spans, triangular mesh.	7-ft. spans.	5 and 6-ft. spans.	Square.	Round.	Twisted.	
Area, in square inches	0.0298	0.0338	0.0288	0.0287	0.0288	0.0662	0.0510	0.0593	0.0099
Percentage of reduction in area	....	....	....	45.2	56.6	55.1	73.8	66.9	....
Percentage of elongation in 8 in.	....	....	....	5.6	4.6	3.6	22.9	8.5	....
Yield point, in pounds per square inch	80 500	70 670	74 500	80 400	82 615	66 330	34 575	50 630	....
Ultimate strength, in pounds per square inch	97 440	77 220	85 870	103 600	104 470	77 200	50 950	62 930	62 600

TABLE 12.—CRUSHING STRENGTH, ETC., OF SLAB CONCRETE.

	Material from Furnald Hall, Columbia University, Used in 1913 Slabs.		Material Used in 1914 Slabs, Cast in Laboratory.		
	6-ft. slabs.	8-ft. slabs.	5-ft. slabs.	6-ft. slabs.	7-ft. slabs.
Mix	1 : 2 : 5	1 : 2 : 5	1 : 2 : 5	1 : 2 : 5	1 : 2 : 5
Weight, in pounds per cubic foot.	113	113	....	....	....
One month test:					
Crushing strength, in pounds per square inch	1 700	1 366	1 389	1 474	1 142
Modulus of elasticity, in pounds per square inch	1 884 000	1 486 000	1 156 000	1 345 000	1 430 000
Two-month test:					
Crushing strength, in pounds per square inch	2 027	2 104	....	....	....
Modulus of elasticity, in pounds per square inch	1 832 000	1 774 000	....	....	....
One-year test:					
Crushing strength, in pounds per square inch	2 570	....	....	....	....
Modulus of elasticity, in pounds per square inch	1 475 000	....	....	....	....

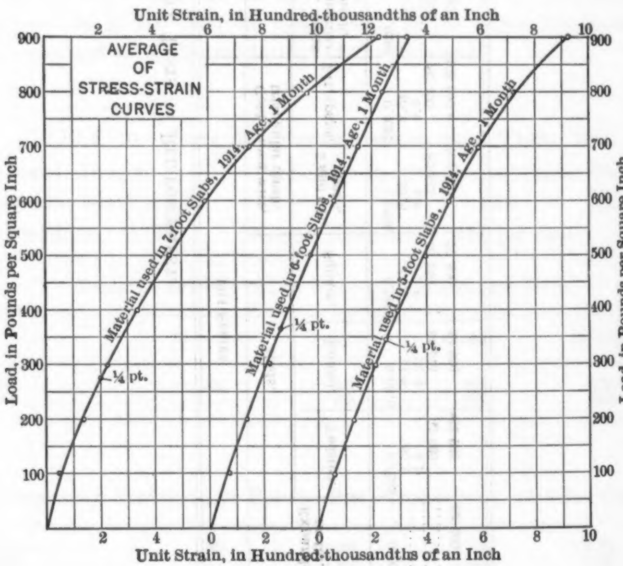


FIG. 31.

APPENDIX III

Typical Slabs.

Full Log Sheets and Curves.

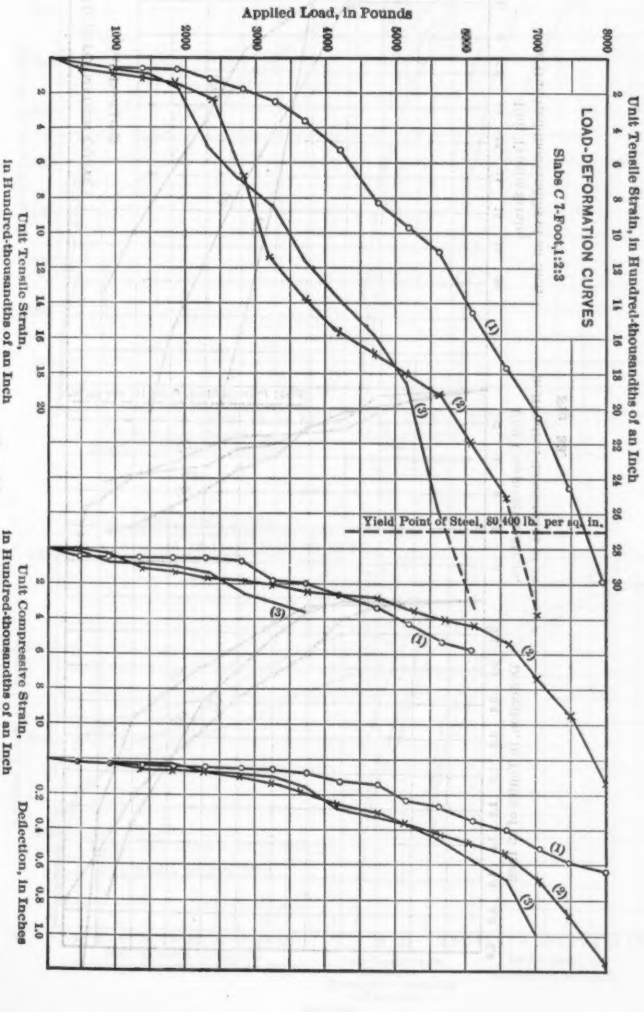


Fig. 32.

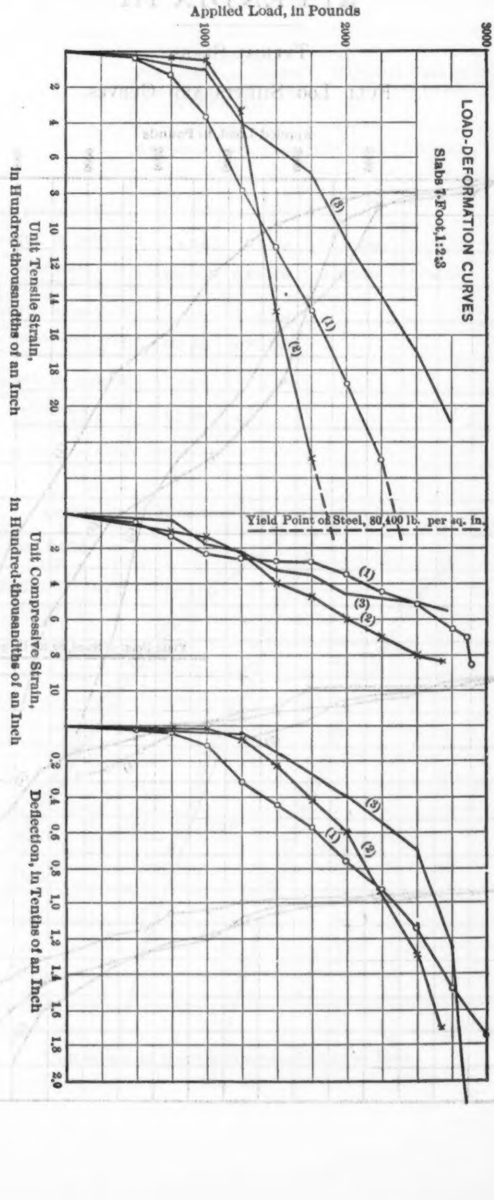

LOAD-DEFORMATION CURVES
Slabs 7-Foot, 1:2:3
Applied Load, in Pounds
1000
2000
3000
Yield Point of Steel, 80,400 lb. per sq. in.
Unit Tensile Strain,
in Hundred-thousandths of an Inch
Unit Compressive Strain,
in Hundred-thousandths of an Inch
Deflection, in Tenths of an Inch
(1)
(2)
(3)

FIG. 33.

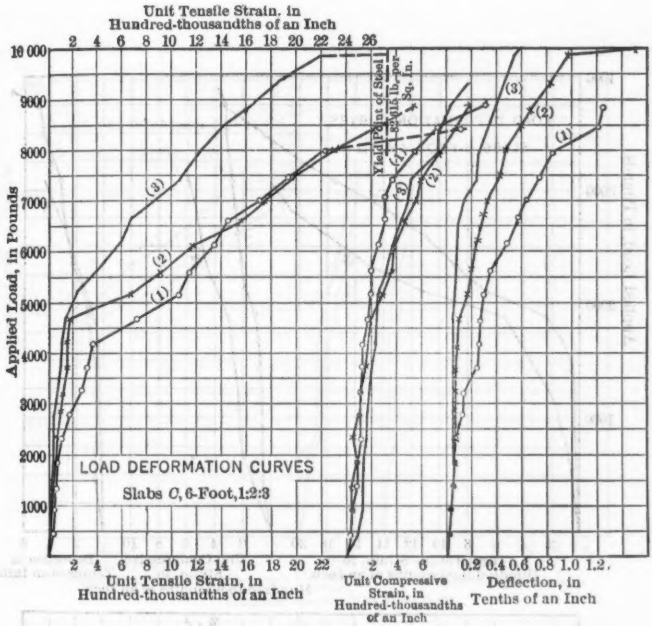

Unit Tensile Strain. in Hundred-thousandths of an Inch
Applied Load, in Pounds
Yield Point of Steel 32615 lb. per Sq. In.
LOAD DEFORMATION CURVES
Slabs C, 6-Foot, 1:2:3
Unit Tensile Strain, in Hundred-thousandths of an Inch
Unit Compressive Strain, in Hundred-thousandths of an Inch
Deflection, in Tenths of an Inch

FIG. 34.

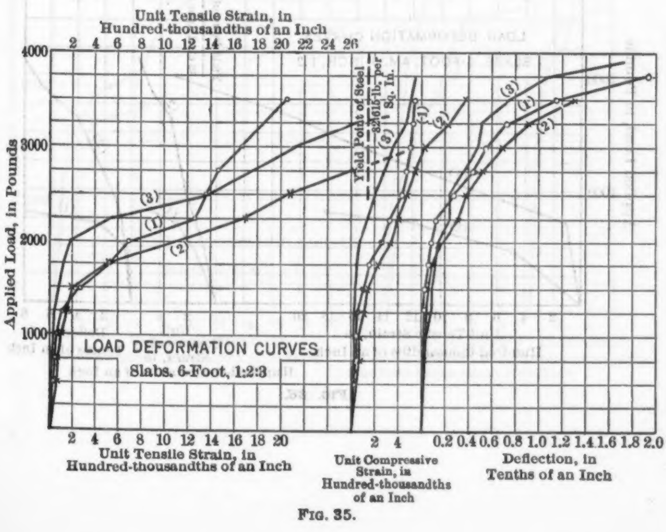

Unit Tensile Strain, in Hundred-thousandths of an Inch
Applied Load, in Pounds
Yield Point of Steel 32615 lb. per Sq. In.
LOAD DEFORMATION CURVES
Slabs. 6-Foot, 1:2:3
Unit Tensile Strain, in Hundred-thousandths of an Inch
Unit Compressive Strain, in Hundred-thousandths of an Inch
Deflection, in Tenths of an Inch

FIG. 35.

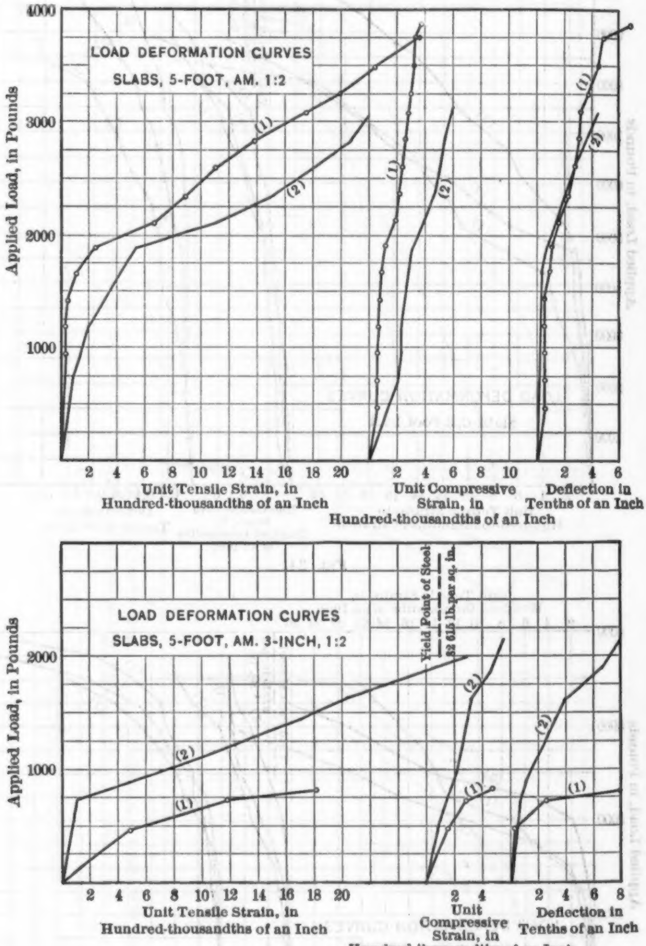

LOAD DEFORMATION CURVES
SLABS, 5-FOOT, AM. 1:2
Applied Load, in Pounds
4000
3000
2000
1000
(1)
(2)
2 4 6 8 10 12 14 16 18 20
Unit Tensile Strain, in Hundred-thousandths of an Inch
2 4 6 8 10
Unit Compressive Strain, in Hundred-thousandths of an Inch
2 4 6
Deflection in Tenths of an Inch
LOAD DEFORMATION CURVES
SLABS, 5-FOOT, AM. 3-INCH, 1:2
Yield Point of Steel
32 615 lb. per sq. in.
Applied Load, in Pounds
2000
1000
2 4 6 8 10 12 14 16 18 20
Unit Tensile Strain, in Hundred-thousandths of an Inch
2 4
Unit Compressive Strain, in Hundred-thousandths of an Inch
2 4 6 8
Deflection in Tenths of an Inch

FIG. 36.

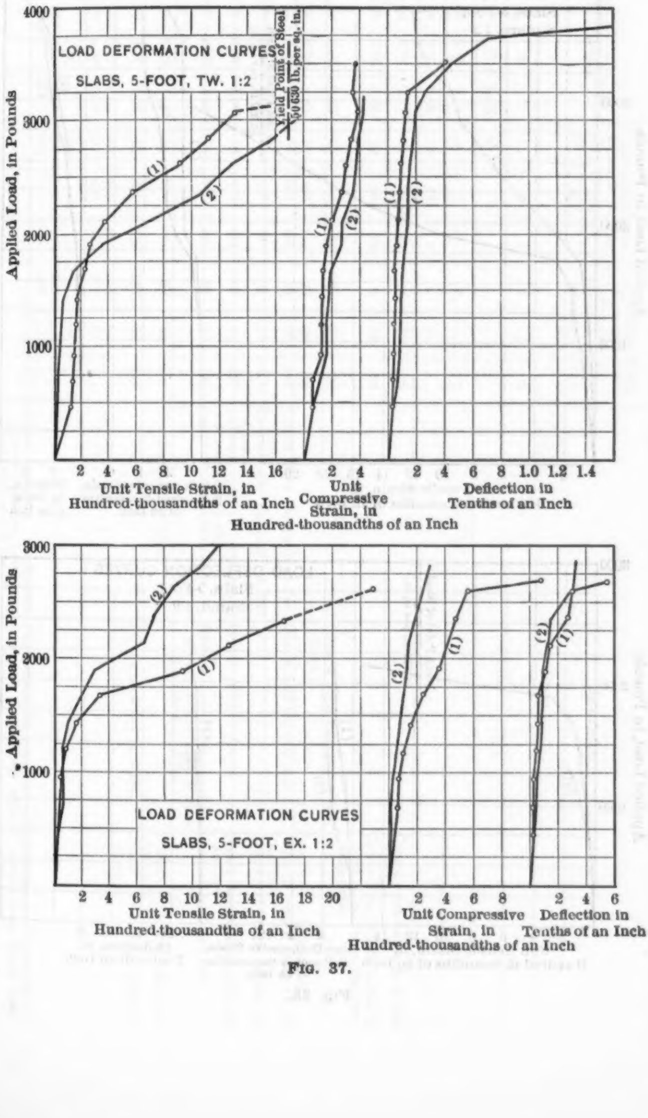
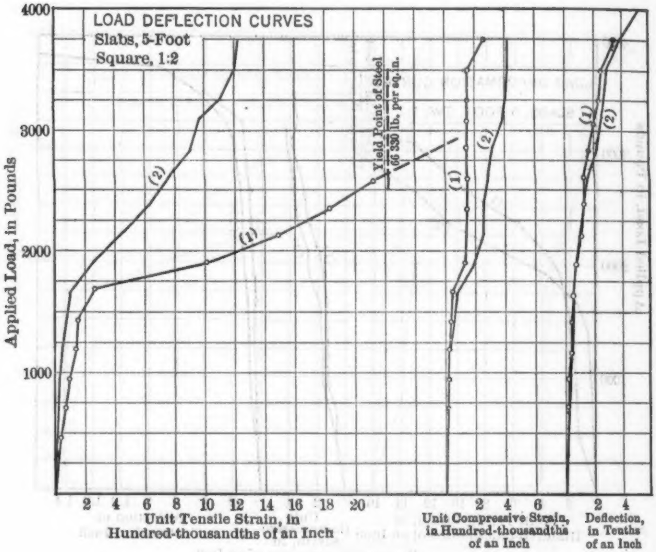
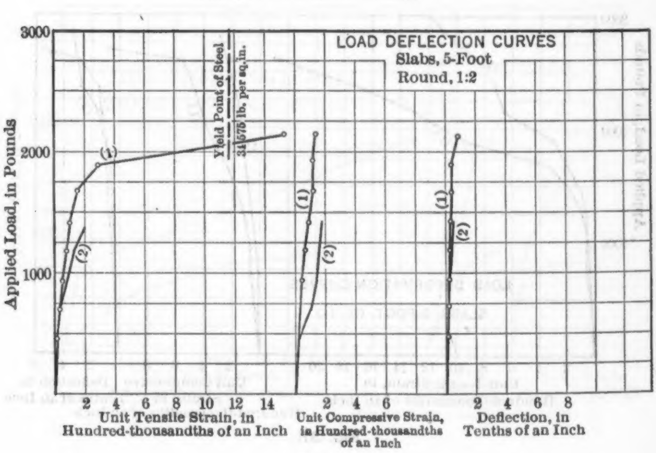

LOAD DEFORMATION CURVES
SLABS, 5-FOOT, TW. 1:2
Yield Point of Steel
50 630 lb. per sq. in.
Applied Load, in Pounds
4000
3000
2000
1000
(1)
(2)
2 4 6 8 10 12 14 16
Unit Tensile Strain, in
Hundred-thousandths of an Inch
2 4
Unit
Compressive
Strain, in
Hundred-thousandths of an Inch
2 4 6 8 1.0 1.2 1.4
Deflection in
Tenths of an Inch
LOAD DEFORMATION CURVES
SLABS, 5-FOOT, EX. 1:2
Applied Load, in Pounds
3000
2000
1000
2 4 6 8 10 12 14 16 18 20
Unit Tensile Strain, in
Hundred-thousandths of an Inch
2 4 6 8
Unit Compressive
Strain, in
Hundred-thousandths of an Inch
2 4 6
Deflection in
Tenths of an Inch

FIG. 37.



LOAD DEFLECTION CURVES
Slabs, 5-Foot
Square, 1:2
Applied Load, in Pounds
4000
3000
2000
1000
Yield Point of Steel
66 330 lb. per sq.in.
(1)
(2)
2 4 6 8 10 12 14 16 18 20
Unit Tensile Strain, in Hundred-thousandths of an Inch
2 4 6
Unit Compressive Strain, in Hundred-thousandths of an Inch
2 4
Deflection, in Tenths of an Inch



LOAD DEFLECTION CURVES
Slabs, 5-Foot
Round, 1:2
Applied Load, in Pounds
3000
2000
1000
Yield Point of Steel
31 575 lb. per sq.in.
(1)
(2)
2 4 6 8 10 12 14
Unit Tensile Strain, in Hundred-thousandths of an Inch
2 4 6
Unit Compressive Strain, in Hundred-thousandths of an Inch
2 4 6 8
Deflection, in Tenths of an Inch

FIG. 38.

TABLE 13.—LOG OF SLAB TESTS.

Total applied load, in pounds.	DEFLECTION, IN INCHES.			AVERAGE UNIT TENSILE STRAIN, IN HUNDRED-THOUSANDTHS OF AN INCH.			AVERAGE UNIT COMPRESSIVE STRAIN, IN HUNDRED-THOUSANDTHS OF AN INCH.			REMARKS.		
	1	2	3	1	2	3	1	2	3	1	2	3
C-7-FT. 1 : 2 : 3-R.												
475	0.02	0.03	0.02	......	7	......	4	......	2			
950	0.03	0.03	0.02	5	10	5	4	8	6			
1 425	0.04	0.05	0.02	6	13	9	4	10	......			
1 900	0.04	0.06	0.03	6	16	19	4	13	10			
2 350	0.04	0.07	0.07	11	22	50	4	17	15			Crack over haunch A-test span.
2 825	0.05	0.09	0.08	19	66	72	6	19	24			Crack 2 in. inside roller B and 4 in. from mid-tension side.
3 250	0.06	0.13	0.10	25	114	82	18	20	29		(Crack over haunch B-test span.)	
3 725	0.08	0.19	0.16	37	139	115	20	23	35			Crack over haunch B.
4 175	0.12	0.25	0.27	54	156	136	26	26	148	Crack over haunch A-test span.	Crack mid-tension side.	Central crack extends across.
4 675	0.15	0.31	0.32	81	167	160	32	29	151	Crack over haunch B-test span.		Crack 4 in. from roller A.
5 175	0.24	0.37	0.36	98	180	204	42	34	169	(Above cracks extending across top of slab.)	Crack under roller A.	Crack at roller extends across.
5 625	0.27	0.43	0.48	110	191	260	52	40	184	Cracks mid-tension side.	(Crack 5 in. from mid-tension side extending across underside test span.)	Crack 5 in. from mid.
6 100	0.34	0.48	0.58	146	219	310	58	42	204	Cracks under rollers.		Crack mid. top anchor-span A.
6 575	0.40	0.55	0.68	178	251	374	148	52	234		First crack 1/16 in. wide.	Crack mid. top anchor-span B.
7 025	0.50	0.67	1.00	205	318	446	154	75	......			
7 475	0.59	0.88	1.86	247	400	605	164	94	......			
7 975	0.63	1.15	......	299	600	......	182	131	......			
8 175	......	1.70	......	354	......	......	......	156	......			
8 450	0.88	......	......	480	......	......	202	......	......			
8 700	1.22	......	......	640	......	......	248	......	......			
										After passing maximum slab carried less load wire fracturing at 7 475 lb. Concrete then crushed over central crack.	Crushing of concrete at crack 5 in. from mid-test span.	Crushing of concrete at crack 5 in. from mid-test span.

TABLE 13.—(*Continued.*)

Total applied load, in pounds.	Deflection, in Inches.			Average Unit Tensile Strain, in Hundred-thousandths of an Inch.			Average Unit Compressive Strain, in Hundred-thousandths of an Inch.			Remarks.		
	1	2	3	1	2	3	1	2	3	1	2	3
C-6-ft. 1 : 2 : 3-R.												
475	0.01	0.02	0.02	4	1	1	2	......	9			
950	0.03	0.04	0.04	5	2	1	4	3	11			
1 425	0.03	0.04	0.04	6	2	3	6	......	11			
1 900	0.05	0.04	0.04	7	5	3	7	6	12			Crack at mid. tension side, test span.
2 350	0.06	0.06	0.04	11	8	3	12	6	14			
2 825	0.15	0.06	0.05	18	9	3	12	12	15			
3 250	0.16	0.06	0.05	25	10	6	13	13	17			
3 725	0.23	0.06	0.06	32	13	8	14	16	20		Crack at roller B—tension side.	
4 175	0.24	0.08	0.06	38	......	8	14	......	21			Crack under roller B, tension side.
4 675	0.25	0.10	0.06	72	15	12	18	19	21	Crack extending across at middle.	Crack over haunch test span.	
5 175	0.30	0.18	0.08	107	66	22	20	29	23		Crack 2 in. from mid. ten. side.	Cracks over both haunches, test span.
5 625	0.35	0.20	0.10	116	94	42	20	36	31	Crack under roller A.		Crack 7 in. from mid. tension side, test span.
6 100	0.47	0.22	0 11	136	116	60	25	37	36			Crack under roller A—tension side.
6 575	0.55	0.28	0 12	146	156	69	30	45	43	Crack over both haunches on test span and top of anchor-spans.	Crack in anchor-span A.	
7 025	0.62	0.32	0.15	171	176	89	30	55	49		Crack in anchor-span B.	
7 475	0.73	0.41	0.22	194	198	107	37	59	52		Crack 2 in. from center.	
7 975	0.85	0.46	0.24	224	230	122	55	75	69	Crack across tension side, 5 in. from middle.		
8 450	1.21	0.60	0.32	331	262	140	72	88	69			
8 675	1.25	......	......	416	......	......	112	......	......			
8 875	......	0.68	0.40	......	295	161	......	99	73			
9 375	......	0.84	0.46	......	......	......	......	122	97			
9 920	......	1.08	0.55	......	......	170	......	171	......			
100 075	......	1.55	......	......	......	......	......	......	......			
10 425	......	......	0.62	......	......	243	......	......	98			
10 920	......	......	0.89	......	......	262	......	......	109			

TABLE 13.—(*Continued.*)

Total applied load, in pounds.	Deflection, in Inches.			Average Unit Tensile Strain, in Hundred-thousandths of an Inch.			Average Unit Compressive Strain, in Hundred-thousandths of an Inch.			Remarks.		
	1	2	3	1	2	3	1	2	3	1	2	3
7-ft. 1 : 2 : 3-F.												
500	0.02	0.01	......	3	0	3	6	0	3			
750	0.04	0.01	......	12	2	7	13	12	......			
1 000	0.12	0.01	......	37	4	10	23	14	18			
1 250	0.32	0.09	0.05	79	35	40	28	26	24	Crack mid-tension side.	Crack mid-tension side.	Crack 2 in. outside roller A.
1 500	0.43	0.21	0.15	111	143	55	28	40	34	Crack extends across.	Crack extends across.	Central crack extending across.
1 750	0.58	0.41	0.27	146	226	70	29	48	36		Crack of one-third point.	
2 000	0.77	0.59	0.40	186	281	108	37	60	45		Crack of one-quarter point.	Crack 3 in. inside roller B.
2 250	0.92	0.93	0.56	228	344	138	44	70	48	Crack of one-third points.		Crack 5 in. from center.
2 500	1.15	1.28	0.70	302	402	170	51	80	52			Crack 7 in. from center.
2 650	......	1.63	......	......	......	......	......	82	......			Crack under roller A.
2 750	1.48	......	1.25	339	......	210	65	......	55			
2 870	1.78	......	2.15	......	......	......	70	......	......			
2 900	......	......	......	......	......	......	84	......	......	Crushed under roller.	Tension failure.	Tension failure.

TABLE 13.—(*Continued.*)

Total applied load, in pounds.	Deflection, in Inches.			Average Unit Tensile Strain, in Hundred-thousandths of an Inch.			Average Unit Compressive Strain, in Hundred-thousandths of an Inch.			Remarks.	
	1	2	3	1	2	3	1	2	3	1	2
6-ft. 1 year.—1 : 2.											
500	0.00	......	0.01	0	......	1	0	......	0		
750	0.01	......	0.01	0	......	2	2	......	1		
1 000	0.01	......	0.03	0	......	3	2	......	2		
1 250	0.01	......	0.03	0	......	4	2	......	5		
1 500	0.02	......	0.05	0	......	5	8	......	5		
1 750	0.02	......	0.06	1	......	6	9	......	6		
2 000	0.02	......	0.08	2	......	8	......	......	12		
2 250	0.12	......	0.26	64	......	15	14	......	12	Crack under roller B.	Cracks 3 in. inside each roller.
2 500	0.27	......	0.42	119	......	163	26	......	21	Crack 1½ in. from mid.	
2 750	0.40	......	0.57	230	......	215	40	......	39		
3 000	0.55	......	0.64	283	......	256	49	......	41	Crack under roller A.	
3 250	0.62	......	0.74	320	......	306	52	......	44	Crack 3 in. from mid.	
3 500	0.75	......	0.89	424	......	406	54	......	52		Cracks 5 in. outside roller A.
3 750	0.95	......	1.31	482	......	......	56	......	70		Cracks 4 in. outside roller B.
3 800	......	......	1.61	......	......	......	......	......	......		
4 000	1.50	......	......	844	......	......	61	......	......		
4 138	......	......	......	......	......	......	......	......	......		

TABLE 13.—(*Continued.*)

Total applied load, in pounds.	Deflection, in Inches.			Average Unit Tensile Strain, in Hundred-thousandths of an Inch.			Average Unit Compressive Strain, in Hundred-thousandths of an Inch.			Remarks.		
	1	2	3	1	2	3	1	2	3	1	2	3
6-ft. 1 : 2 : 3.												
500	0.00	0.00	0.00	2	4	1	2	0	1			
750	0.00	0.01	0.01	3	6	2	4	5	3			
1 000	0.00	0.02	0.03	5	8	3	4	6	3			
1 250	0.01	0.02	0.03	10	......	4	7	14	4		Crack mid. tension side.	
1 500	0.01	0.03	0.05	25	21	6	11	16	6			
1 750	0.01	0.07	0.06	54	53	11	14	22	7		Crack 7 in. from middle.	
2 000	0.01	0.15	0.10	70	......	28	29	32	13	Crack tension side, mid.	Crack under roller B.	
2 250	0.01	0.30	0.12	128	166	56	32	39	19	Crack tension side, under both rollers.		Crack 10 in. from mid. and at mid.
2 500	0.26	0.37	0.22	137	204	140	41	46	23			Crack 8 in. from mid.
2 750	0.42	0.51	0.23	146	263	171	48	52	28	Crack 7 in. from middle.		Crack under roller A.
3 000	0.57	0.69	0.38	168	389	211	51	62	36	Crack 7 in. from middle, opposite end.	Crack outside roller A.	
3 250	0.74	0.91	0.50	......	453	262	53	81	47	Crack 7 in. outside roller A.		
3 500	1.18	1.31	0.52	203	654	318	56	95	51	" 8 in. " " B.		
3 550	......	......	0.74	......	......	......	......	......	......			
3 750	1.97	......	1.07	......	......	398	......	......	55	Fractured wire.		
3 900	......	......	1.72	......	......	......	......	......	......			

TABLE 13.—(*Continued.*)

Applied load, in pounds.	Average Deflection, in Inches.		Average Unit Tensile Strain, in Hundred-thousandths of an Inch.		Average Unit Compressive Strain, in Hundred-thousandths of an Inch.		Remarks.	
	1	2	1	2	1	2	1	2
5-ft. Ex.–1 : 2.								
475	0.02	0.02	....	1	....	1		
725	0.02	0.03	....	4	6	1		
950	0.02	0.04	3	4	7	2		
1 200	0.04	0.06	7	5	10	5		
1 425	0.05	0.07	16	10	11	7		
1 675	0.05	0.09	33	19	23	10		
1 900	0.11	0.10	94	28	35	15		
2 125	0.15	0.13	126	64	....	16	Crack at mid. tension side.	
2 350	0.24	0.14	165	74	47	19		
2 600	0.30	0.30	228	87	55	23		Crack 3 in. from mid.
2 690	0.55	....	....	....	107	....		
2 825	....	0.83	....	108	....	27		
3 005	....	....	....	118	....	23		

TABLE 13.—(*Continued.*)

Applied load, in pounds.	Average Deflection, in Inches.		Average Unit Tensile Strain, in Hundred-thousandths of an Inch.		Average Unit Compressive Strain, in Hundred-thousandths of an Inch.		Remarks.	
	1	2	1	2	1	2	1	2
5–ft. Am. 1 : 2.								
475	0.04	0.01	....	6	5	14		Tension reading taken on a 12-in. gauge length ; compression on a 10-in. gauge length.
725	0.04	0.02	....	8	5	20		
950	0.05	0.02	1	14	5	23		
1 200	0.05	0.03	2	19	7	23		
1 425	0.06	0.04	3	30	8	25		
1 675	0.08	0.05	10	42	9	28		
1 900	0.10	0.08	24	72	12	31		
2 125	0.15	0.14	65	110	19	40	Crack under roller B, tension side.	Crack under roller A, tension side.
2 350	0.22	0.20	89	149	21	47		Crack at mid., tension side.
2 600	0.26	0.26	110	179	23	52		Crack 3 in. outside roller B, tension side.
2 825	0.29	0.35	138	206	26	55		
3 050	0.32	0.44	176	220	29	60		
3 250	0.37	....	199	....	31	....	Crack 4 in. outside roller A, also at mid.	
3 500	0.44	....	225	....	34	....	Crack 8 in. from mid.	
3 725	0.48	....	255	....	34	....		
3 875	0.72	....	298	....	39	....		
5–ft. 3–in. Am. 1 : 2.								
475	0.02	0.05	48	6	16	8		
725	0.24	0.07	118	10	28	14	Crack 2 in. from mid., also 2 n. outside roller A.	
81	0.82	....	181	....	47	....	Crack 1 in. outside roller A.	
955	....	0.13	....	65	....	26	Crack under roller B.	Crack under both rollers, tension side.
1 200	....	0.22	....	116	....	....		Crack at middle.
1 420	....	0.30	....	160	....	28		
1 675	....	0.40	....	201	....	34		
1 905	....	0.66	....	260	....	46		
2 120	....	0.81	....	362	....	56		
2 215	....	....	....	410	....	56		

TABLE 13.—(*Continued.*)

Applied load, in pounds.	Average Deflection, in Inches. 1	Average Deflection, in Inches. 2	Average Unit Tensile Strain, in Hundred-thousandths of an Inch. 1	Average Unit Tensile Strain, in Hundred-thousandths of an Inch. 2	Average Unit Compressive Strain, in Hundred-thousandths of an Inch. 1	Average Unit Compressive Strain, in Hundred-thousandths of an Inch. 2	Remarks. 1	Remarks. 2
5-ft. Rd.–1 : 2 : 3.								
475	0.01	....	1	1	0	2		
725	0.01	....	2	2	0	6		
950	0.02	0.01	4	5	3	9		
1 200	0.02	0.03	6	8	6	11		
1 425	0.03	0.04	9	20	9	13		Crack 7 in. from middle.
1 575								
1 675	0.03	....	16	....	12	....		
1 900	0.04	....	28	....	12	....		
2 125	0.06	....	154	....	14	....	Crack 2 in. from middle. Bond failure.	Bond failure.
7-ft. Am. Special.								
475	....	....	....	....	....	....		
950	....	....	....	....	....	....		
1 425	....	....	....	....	....	....		
1 900	....	0.01	....	....	....	....		
2 125	....	....	....	....	....	....	Crackling sounds at free end.	
2 850	....	0.04	....	....	....	....		
2 825	....	0.08	....	....	....	....		
3 250	....	0.12	....	....	....	....	Crack across fixed end test slab 4 in. from beam.	
3 725	....	0.22	....	....	....	....	Cracks under roller at free end and at free haunch fillet. Crack 6 in. toward free end from mid Crack between mid. and roller at free end.	
4 175	....	0.56	....	....	....	....	Crack under roller at fixed end. Free end haunch crack open 1/16 in. Overhang at top of beam at free end lifting.	
4 675	....	0.63	....	....	....	....	Crushing of concrete under roller at free end.	
							Wires fractured in test span and crack appeared 1 ft. from beam on top of unloaded span.	

TABLE 13.—(*Continued.*)

Applied load, in pounds.	Average Deflection, in Inches.		Average Unit Tensile Strain, in Hundred-thousandths of an Inch.		Average Unit Compressive Strain, in Hundred-thousandths of an Inch.		Remarks	
	1	2	1	2	1	2	1	2
5-ft. Tw. 1 : 2								
475	0.02	0.02	14	2	6	6		
725	0.02	0.03	16	2	6	11		
920	0.02	0.04	16	4	12	15		
1 200	0.03	0.05	17	6	12	15		
1 425	0.04	0.05	18	9	12	18		
1 675	0.04	0.07	22	15	13	19		
1 900	0.04	0.10	26	35	16	25		
2 125	0.06	0.12	38	74	20	26		
2 350	0.06	0.12	58	110	26	33		
2 600	0.08	0.14	92	131	30	37		
2 825	0.10	0.17	112	158	33	38		Cracks at middle and under roller A.
3 050	0.12	0.20	130	187	39	41	Cracks 2 in. outside roller A.	
3 250	0.15	0.25	190	287	36	41	Cracks 2 in. inside roller B.	
3 500	0.42	0.43	190	529	37	24	Cracks 3 in. inside roller A.	
3 545	....	0.74	....	....	....	8		Crack 5 in. from center.
3 590	....	1.62	....	....	....	....		
5-ft. Sq. 1 : 2								
475	....	....	3	1	0	....		
725	0.01	0.01	5	2	1	....		
950	0.01	0.02	8	2	1	....		
1 200	0.02	0.02	13	4	2	....		
1 425	0.02	0.03	16	6	4	5		
1 675	0.03	0.04	26	9	6	9		
1 900	0.05	0.04	101	24	13	18		
2 125	0.10	0.09	148	42	....	23		
2 350	0.13	0.12	182	62	15	23		
2 600	0.16	0.16	212	76	15	27		
2 825	0.18	0.22	236	90	15	32		
3 050	0.20	0.24	266	98	15	39		
3 250	0.25	0.27	....	110	15	34	Crack at middle.	Crack 1 in. outside roller B and at middle.
3 500	0.28	0.32	....	120	....	41	Crack 5 in. outside roller A and 2 in. outside roller B.	Crack at roller A.
3 725	0.33	0.38	....	124	25	41		
3 825	....	0.48	....	....	....	37		Crack 7 in outside roller B.

TABLE 13.—(*Continued.*)

Applied load, in pounds.	DEFLECTION, IN INCHES.							
	1	2						
5-FT. PLAIN.								
100	....	....	....	....	....	....		
200	....	....	....	....	....	....		
300	....	....	....	....	....	....		
400	....	....	....	....	....	....		
500	....	....	....	....	....	....		
600	....	....	....	....	....	....		
700	0.001	0.001	....	....	....	....		
800	0.0015	0.001	....	....	....	....		
900	0.002	0.0035	....	....	....	....		
1 000	....	0.004	....	....	....	....		
1 100	0.010	0.0085	....	....	....	....		
1 200	0.012	0.0115	....	....	....	....		
1 300	0.015	0.014	....	....	....	....		
1 400	0.019	0.0175	....	....	....	....		
1 500	....	0.0275	....	....	....	....		

APPENDIX IV

TESTS OF VARIOUS MIXTURES OF CINDER CONCRETE CAST AND TESTED UNDER IDENTICAL CONDITIONS.

Method.—Machine-mixed concrete of medium consistency was cast in the form of standard cylinders, in sets of four. They were placed in water after 48 hours, removed from water 48 hours before testing, and faced with plaster of Paris; age, 28 days.

Materials.—

Dragon Portland cement.

Long Island sand.

Anthracite clinker, Columbia University power plant.

Results.—

TABLE 14.—ULTIMATE COMPRESSIVE STRENGTH, IN POUNDS PER SQUARE INCH, AT 28 DAYS.

Mixture.	Cinders as received.	Cinders finer than ¼-in. screened out.	Weight, in Pounds per Cubic Foot.	
			Unscreened.	Screened.
1 : 2 : 4	975	779	114	113
1 : 2 : 5	787	585	115	110
1 : 3 : 6	708	670	113	112
1 : 7	382	...	83	...

Remarks.—The results in Table 14, inclusive of weights, must be considered only as comparative, because, not only were the specimens stored in water, but the consistency also was rather dry.

APPENDIX V

Derivations of Expressions for Bending Moments and Shears; and Miscellaneous.

Uniform Load.—Three Spans, Fig. 39.—

$$M_1 l_1 + 2 M_2 (l_1 + l_2) + M_3 l_2 = -6\left(\frac{s_1 y_1}{1} + \frac{s_2 y_2}{2}\right) \quad \text{.....(1)}$$

where s_1 and s_2 are bending moment areas for simple span, and y_1 and y_2 are distances to supports from centers of gravity.

$$2 M_2 \left(\frac{l}{2} + l\right) + M_3 l = -\frac{1}{32} pl^3 - \frac{1}{4} pl^3$$

as $M_1 = M_4 = 0$ (free ends),

$$\text{or } 3 M_2 l + M_3 l = -\frac{9}{32} pl^3 \quad \text{.....(2)}$$

$$\text{Similarly, } M_2 l + 3 M_3 l = -\frac{9}{32} pl^3 \quad \text{.....(3)}$$

Solving Equations (2) and (3),

$$M_2 = M_3 = -\frac{9}{128} pl^2 = -\frac{1}{14.2} pl^2 \quad \text{.....(4)}$$

Taking moments of Span 1 about Support 2,

$$R_1 \times \frac{1}{2} l - \frac{1}{2} pl \times \frac{1}{4} l = -\frac{9}{128} pl^2.$$

$$\text{Whence} \qquad R_1 = +\frac{7}{64} pl. \quad \text{.....(5)}$$

Taking moments of Span 1 about Support 1,

$$R_2 \times \frac{1}{2} l - \frac{9}{128} pl^2 - \frac{1}{8} pl^2 = 0.$$

$$\text{Whence} \qquad R_2' = +\frac{25}{64} pl. \quad \text{.....(6)}$$

$$\text{also} \qquad R_2'' = +\frac{1}{2} pl \quad \text{.....(7)}$$

Taking moments about the center of Span 2,

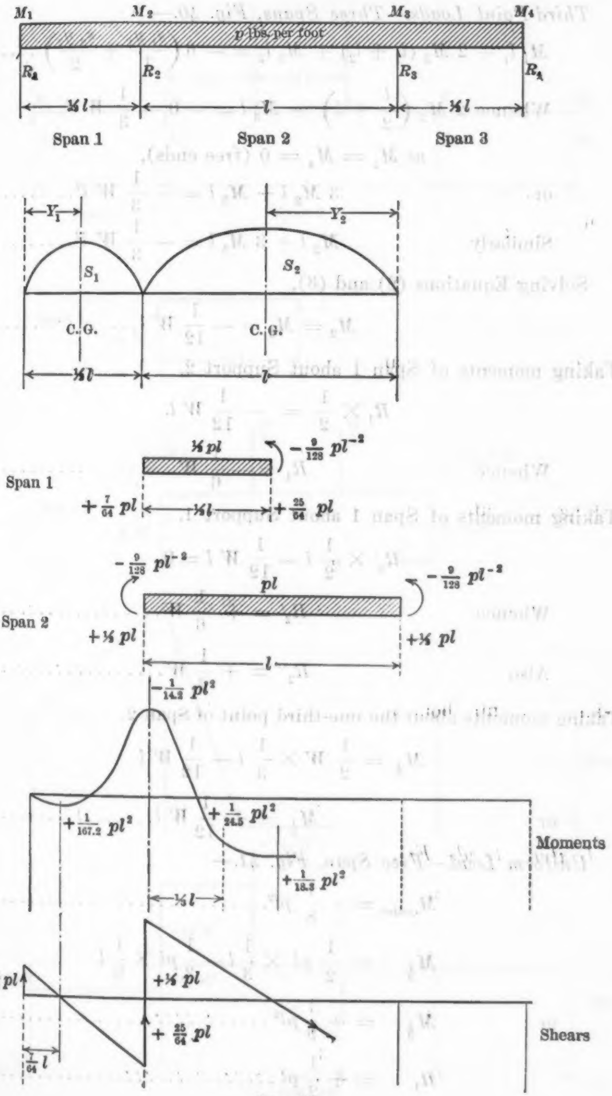
$$M_c = \frac{1}{2} pl \times \frac{1}{2} l - \frac{9}{128} pl^2 - \frac{1}{8} pl^2$$

$$M_c = +\frac{7}{128} pl^2 = +\frac{1}{18.3} pl^2 \quad \text{.....(8)}$$

Taking moments about the one-third point of Span 2,

$$M_{\frac{1}{3}} = \frac{1}{2} pl \times \frac{1}{3} l - \frac{9}{128} pl^2 - \frac{1}{3} pl \times \frac{1}{6} l$$

$$M_{\frac{1}{3}} = +\frac{47}{1\,152} pl^2 = +\frac{1}{24.5} pl^2 \quad \text{.....(9)}$$



M_1 M_2 M_3 M_4
p lbs. per foot
R_1 R_2 R_3 R_4
½ l
l
½ l
Span 1
Span 2
Span 3
Y_1
Y_2
S_1
S_2
C. G.
C. G.
½ l
l
Span 1
½ pl
$-\frac{9}{128}\,pl^{-2}$
$+\frac{7}{64}\,pl$
½ l
$+\frac{25}{64}\,pl$
Span 2
$-\frac{9}{128}\,pl^{-2}$
pl
$-\frac{9}{128}\,pl^{-2}$
+½ pl
+½ pl
l
$-\frac{1}{14.2}\,pl^2$
$+\frac{1}{167.2}\,pl^2$
$+\frac{1}{24.5}\,pl^2$
$+\frac{1}{18.3}\,pl^2$
Moments
¼ l
$+\frac{7}{64}\,pl$
+½ pl
$+\frac{25}{64}\,pl$
$\frac{7}{64}\,l$
Shears

FIG. 39.

Third-Point Loads.—Three Spans, Fig. 40.—

$$M_1 l_1 + 2 M_2 (l_1 + l_2) + M_3 l_2 = -6\left(\frac{s_1 y_1}{1} + \frac{s_2 y_2}{2}\right) \text{......(1)}$$

Whence $2 M_2 \left(\frac{l}{2} + l\right) + M_3 l = -0 - \frac{1}{3} W l^2$

as $M_1 = M_4 = 0$ (free ends),

or $$3 M_2 l + M_3 l = -\frac{1}{3} W l^2 \text{..........(2)}$$

Similarly, $$M_2 l + 3 M_3 l = -\frac{1}{3} W l^2 \text{..........(3)}$$

Solving Equations (2) and (3),

$$M_2 = M_3 = -\frac{1}{12} W l \text{.................(4)}$$

Taking moments of Span 1 about Support 2,

$$R_1 \times \frac{1}{2} = -\frac{1}{12} W l.$$

Whence $$R_1 = -\frac{1}{6} W \text{....................(5)}$$

Taking moments of Span 1 about Support 1,

$$R_2' \times \frac{1}{2} l - \frac{1}{12} W l = 0.$$

Whence $$R_2' = +\frac{1}{6} W \text{....................(6)}$$

Also $$R_2'' = +\frac{1}{2} W \text{....................(7)}$$

Taking moments about the one-third point of Span 2,

$$M_{\frac{1}{3}} = \frac{1}{2} W \times \frac{1}{3} l - \frac{1}{12} W l$$

or $$M_{\frac{1}{3}} = +\frac{1}{12} W l \text{.................(8)}$$

Uniform Load.—Free Span, Fig. 41.—

$$M_{center} = +\frac{1}{8} pl^2 \text{..............................(1)}$$

$$M_{\frac{1}{3}} = \frac{1}{2} pl \times \frac{1}{3} l - \frac{1}{3} pl \times \frac{1}{6} l$$

or $$M_{\frac{1}{3}} = +\frac{1}{9} pl^2 \text{..............................(2)}$$

$$R_1 = +\frac{1}{2} pl \text{..............................(3)}$$

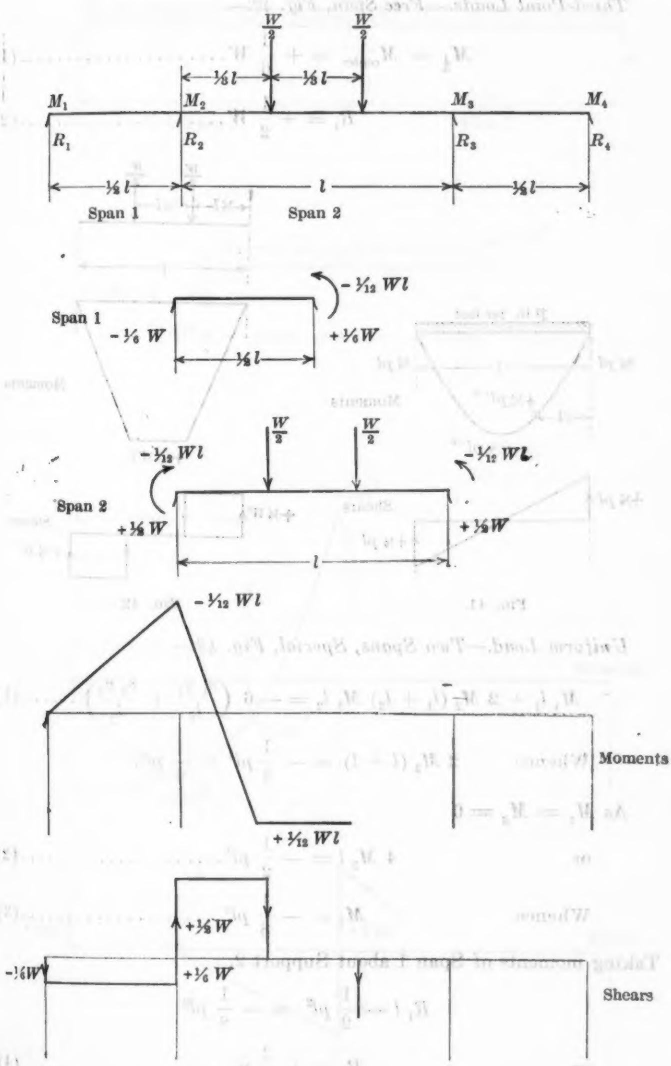

W/2
W/2
1/3 l
1/3 l
M1
M2
M3
M4
R1
R2
R3
R4
1/2 l
l
1/2 l
Span 1
Span 2
- 1/12 Wl
Span 1
- 1/6 W
+ 1/6 W
1/2 l
W/2
W/2
- 1/12 Wl
- 1/12 Wl
Span 2
+ 1/2 W
+ 1/2 W
l
- 1/12 Wl
Moments
+ 1/12 Wl
+ 1/2 W
- 1/6 W
+ 1/6 W
Shears

FIG. 40.

Third-Point Loads.—Free Span, Fig. 42.—

$$M_{\frac{1}{3}} = M_{center} = +\frac{1}{6}W \ldots\ldots\ldots\ldots (1)$$

$$R_1 = +\frac{1}{2}W \ldots\ldots\ldots\ldots (2)$$

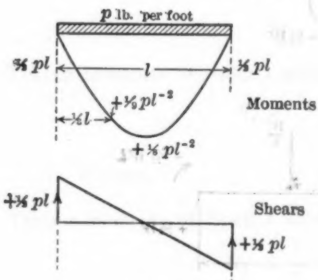


FIG. 41.

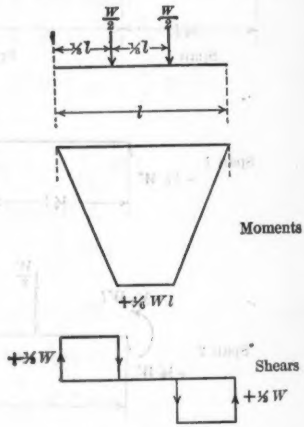


FIG. 42.

Uniform Load.—Two Spans, Special, Fig. 43.—

$$M_1 l_1 + 2 M_2 (l_1 + l_2) M_3 l_2 = -6\left(\frac{s_1 y_1}{l_1} + \frac{s_2 y_2}{l_2}\right) \ldots\ldots (1)$$

Whence $\quad 2 M_2 (l + l) = -\frac{1}{4} pl^3 - \frac{1}{4} pl^3.$

As $M_1 = M_3 = 0$

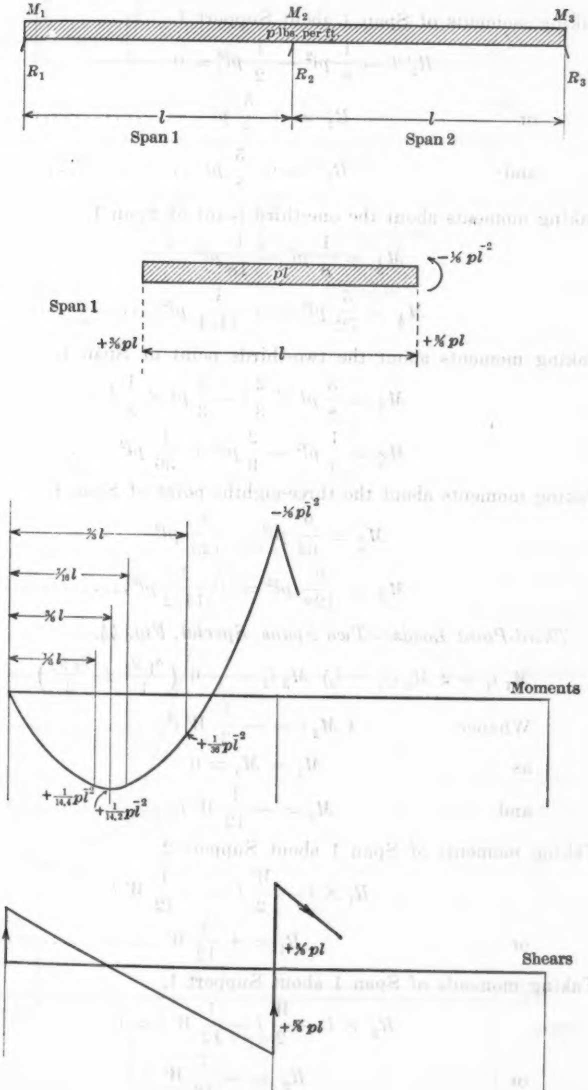
or $$4 M_2 l = -\frac{1}{2} pl^3 \ldots\ldots\ldots\ldots (2)$$

Whence $$M_2 = -\frac{1}{8} pl^2 \ldots\ldots\ldots\ldots (3)$$

Taking moments of Span 1 about Support 2,

$$R_1 l - \frac{1}{2} pl^2 = -\frac{1}{8} pl^2$$

or $$R_1 = +\frac{3}{8} pl \ldots\ldots\ldots\ldots (4)$$



M1
M2
M3
p lbs. per ft.
R1
R2
R3
l
l
Span 1
Span 2
pl
Span 1
+⅜pl
+⅝pl
l
Moments
Shears
+⅜pl
+⅝pl

FIG. 43.

Taking moments of Span 1 about Support 1,

$$R_2' \, l - \frac{1}{8} pl^2 - \frac{1}{2} pl^2 = 0$$

or $$R_2' = + \frac{5}{8} pl \quad \text{.......................(5)}$$

and $$R_2'' = + \frac{5}{8} pl \quad \text{.......................(6)}$$

Taking moments about the one-third point of Span 1,

$$M_{\frac{1}{3}} = \frac{1}{8} pl^2 - \frac{1}{18} pl^2$$

$$M_{\frac{1}{3}} = \frac{5}{72} pl^2 = + \frac{1}{14.4} pl^2 \quad \text{...................(7)}$$

Taking moments about the two-thirds point of Span 1,

$$M_{\frac{2}{3}} = \frac{3}{8} pl \times \frac{2}{3} l - \frac{2}{3} pl \times \frac{1}{3} l$$

$$M_{\frac{2}{3}} = \frac{1}{4} pl^2 - \frac{2}{9} pl^2 + \frac{1}{36} pl^2 \quad \text{...............(8)}$$

Taking moments about the three-eighths point of Span 1,

$$M_{\frac{3}{8}} = \frac{9}{64} pl^2 - \frac{9}{128} pl^2$$

$$M_{\frac{3}{8}} = \frac{9}{128} pl^2 = + \frac{1}{14.2} pl^2 \quad \text{.................(9)}$$

Third-Point Loads.—Two Spans, Special, Fig. 44.—

$$M_1 l_1 + 2 M_2 (l_1 + l_2) M_3 l_2 = - 6 \left(\frac{s_1 y_1}{1} + \frac{s_2 y_2}{2} \right) \quad \text{.......(1)}$$

Whence $$4 M_2 l = - \frac{1}{3} W l^2$$

as $$M_1 = M_3 = 0$$

and $$M_2 = - \frac{1}{12} W l \quad \text{.......................(2)}$$

Taking moments of Span 1 about Support 2,

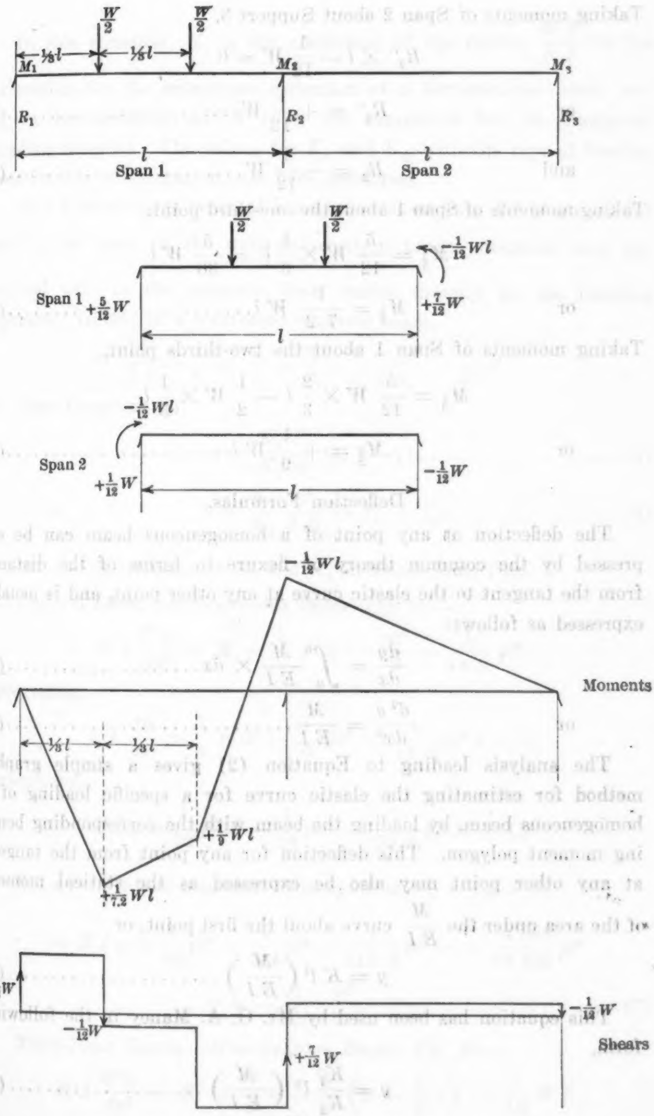
$$R_1 \times l - \frac{W}{2} l = - \frac{1}{12} W l$$

or $$R_1 = + \frac{5}{12} W \quad \text{.......................(3)}$$

Taking moments of Span 1 about Support 1,

$$R_2' \times l - \frac{W}{2} l - \frac{1}{12} W l = 0$$

or $$R_2' = + \frac{7}{12} W$$



W/2
W/2
1/3 l
1/3 l
M1
M2
M3
R1
R2
R3
l
l
Span 1
Span 2
W/2
W/2
-1/12 Wl
Span 1
+5/12 W
+7/12 W
l
-1/12 Wl
Span 2
+1/12 W
-1/12 W
l
-1/12 Wl
Moments
1/3 l
1/3 l
+1/9 Wl
+1/7.2 Wl
+5/12 W
-1/12 W
-1/12 W
Shears
+7/12 W

FIG. 44.

Taking moments of Span 2 about Support 3,

$$R_2'' \times l - \frac{1}{12} W = 0$$

or

$$R_2'' = + \frac{1}{12} W \quad \text{....(4)}$$

and

$$R_3 = - \frac{1}{12} W \quad \text{....(5)}$$

Taking moments of Span 1 about the one-third point,

$$M_{\frac{1}{3}} = \frac{5}{12} W \times \frac{1}{3} l = \frac{5}{36} W l$$

or

$$M_{\frac{1}{3}} = \frac{1}{7.2} W l \quad \text{....(6)}$$

Taking moments of Span 1 about the two-thirds point,

$$M_{\frac{2}{3}} = \frac{5}{12} W \times \frac{2}{3} l - \frac{1}{2} W \times \frac{1}{3} l$$

or

$$M_{\frac{2}{3}} = + \frac{1}{9} W l \quad \text{....(7)}$$

Deflection Formulas.

The deflection at any point of a homogeneous beam can be expressed by the common theory of flexure in terms of the distance from the tangent to the elastic curve at any other point, and is usually expressed as follows:

$$\frac{dy}{dx} = \int_a^b \frac{M}{E\,I} \times dx \quad \text{....(1)}$$

or

$$\frac{d^2 y}{dx^2} = \frac{M}{E\,I} \quad \text{....(2)}$$

The analysis leading to Equation (2) gives a simple graphic method for estimating the elastic curve for a specific loading of a homogeneous beam, by loading the beam with the corresponding bending moment polygon. This deflection for any point from the tangent at any other point may also be expressed as the statical moment of the area under the $\frac{M}{E\,I}$ curve about the first point, or

$$y = K\,l^2 \left(\frac{M}{E\,I}\right) \quad \text{....(3)}$$

This equation has been used by Mr. G. A. Maney in the following form,

$$y = \frac{K_1}{K_2} l^2 \left(\frac{M}{E\,I}\right) \quad \text{....(4)}$$

In this equation, K_1 is the coefficient of the factor, $\frac{W l^3}{E I}$, in the expression for the maximum deflection of a homogeneous beam, and K_2 is the coefficient of $W\,l$, in the expression for the maximum bending moment. The values for K_1 and K_2, with the type of loading used in this investigation, are here developed.

In a homogeneous beam of constant cross-section, $E\,I$ is a constant and $\frac{e}{c}$, the ratio of the unit deformation to the distance from the neutral axis to the extreme fiber, varies directly as the bending moment. Hence, in a reinforced concrete beam,

$$\frac{M}{E\,I} = \frac{e}{c} = \frac{e_c}{c_c} = \frac{e_s}{c_s} = \frac{e_c + e_s}{c_c + c_s} = \frac{e_c + e_s}{d}$$

or, from Equation (4),

$$y = \frac{K_1}{K_2}\,\frac{l^2}{d}\,(e_c + e_s) \quad \text{.....................(5)}$$

or

$$y = \frac{K_1}{K_2}\,\frac{l^2}{d\,E_s}\,(n\,f_c + f_s) \quad \text{.................(6)}$$

Derivation of K_1 and K_2 on Three-Span Test.

Uniform Load.—Homogeneous Beam, Fig. 39.—

$$E\,I\,\frac{d^2 y}{dx^2} = M = \frac{1}{2}\,pl\,x - \frac{px^2}{2} - \frac{1}{14.2}\,pl^2.$$

Integrating,

$$E\,I\,\frac{dy}{dx} = \frac{1}{4}\,pl\,x^2 - \frac{1}{6}\,px^3 - \frac{1}{14.2}\,pl^2\,x + C_1.$$

Integrating,

$$E\,I\,y = \frac{1}{12}\,pl\,x^3 - \frac{1}{24}\,px^4 - \frac{1}{28.4}\,pl^2\,x^2 + C_2.$$

But $x = \frac{1}{2}\,l$, and constants of integration, $C_1 = C_2 = 0$,

$$\text{or } E\,I\,y = \frac{1}{96}\,pl^4 - \frac{1}{384}\,pl^4 - \frac{1}{113.6}\,pl^4 = -\frac{4\,608}{43\,622}\,pl^4$$

$$y = -\frac{1}{910}\,\frac{pl^4}{E\,I} \quad \text{........................(7)}$$

Third-Point Loads.—Homogeneous Beam, Fig. 40.—

$$E\,I\,\frac{d^2 y}{dx^2} = M = \frac{W}{2}\,x - \frac{W}{2}\left(x - \frac{1}{3}\,l\right) - \frac{1}{12}\,W\,l$$

Integrating,

$$E\,I\,\frac{dy}{dx} = \frac{1}{12}\,W\,l\,x + C_1.$$

Integrating,

$$E\,I\,y = \frac{1}{24}\,W\,l\,x^2 + C_2.$$

But $\qquad x = \frac{1}{2}\,l$ and $C_1 = C_2 = 0$

or $$y = \frac{1}{96}\,\frac{W\,l^3}{E\,I} \qquad (8)$$

From Equations (7) and (8), $K_1 = \left(\frac{1}{96} - \frac{1}{910}\,\frac{W'}{W}\right)$ ......(9)

The value for K_2 is obtained from the bending moment formulas previously derived; W' = dead weight of slab.

Simple Span Test.

Uniform Load.—Homogeneous Beam, Fig. 41.—

$$y = \frac{5}{384}\,\frac{pl^4}{E\,I} \qquad (1)$$

and $$M = \frac{1}{8}\,pl^2 \qquad (2)$$

Third-Point Loads.—

$$y = \frac{23}{1\,296}\,\frac{W\,l^3}{E\,I} \qquad (3)$$

$$M = \frac{1}{6}\,W\,l \qquad (4)$$

From Equations (1) and (3),

$$K_1 = \left(\frac{1}{56.4} + \frac{1}{76.8}\,\frac{W'}{W}\right) \qquad (5)$$

TABLE 15.—Bending Moment Factors.

Span and end condition.	Reinforcement.		Load, in Pounds.		Equivalent uniform load, $\frac{4}{3}W + W'$, in pounds.	Analysis of Slab at Deformation Within First Stage.										
	Nominal area, in square inches.	Effective area, in square inches.	W	W'		Measured stress, in pounds per square inch.		Computed stress.			Deflection, in inches.		Derived B. M. coefficient.			
								Common theory.		Deflection.			Method.			
						Steel.	Concrete.	Steel.	Concrete.	Steel.	Measured.	Computed.	1	2	3	4
C—7-ft., No. 1...........	0.202	0.173	2 125	481	3 318	3 000	117	35 500	570	3 690	0.045	0.040	$\frac{1}{78}$	$\frac{1}{190}$	$\frac{1}{155}$	$\frac{1}{21}$
C—7-ft., No. 2...........	0.202	0.173	2 400	473	3 673	9 000	520	41 000	675	......	0.09	0.17	$\frac{1}{21}$	$\frac{1}{72}$	....	$\frac{1}{21}$
C—7-ft., No. 3...........	0.202	0.173	2 200	481	3 414	12 000	438	36 000	580	6 720	0.13	0.17	$\frac{1}{22}$	$\frac{1}{49}$	$\frac{1}{88}$	$\frac{1}{21}$
7-ft., No. 1...........	0.202	0.173	880	478	1 770	7 500	537	35 900	418	927	0.10	0.15	$\frac{1}{7}$	$\frac{1}{41}$	$\frac{1}{33}$	$\frac{1}{8}$
7-ft., No. 2...........	0.202	0.173	1 268	525	2 215	12 800	772	49 200	800	1 935	0.13	0.21	$\frac{1}{7}$	$\frac{1}{26}$	$\frac{1}{17}$	$\frac{1}{8}$
7-ft., No. 3...........	0.202	0.173	1 056	501	1 910	5 000	553	41 250	666	......	0.04	0.13	$\frac{1}{9}$	$\frac{1}{62}$	....	$\frac{1}{8}$
C—6-ft., No. 1...........	0.187	0.173	4 300	386	6 126	14 050	475	60 500	1 134	24 060	0.24	0.17	$\frac{1}{37}$	$\frac{1}{69}$	$\frac{1}{41}$	$\frac{1}{20}$
C—6-ft., No. 2...........	0.187	0.173	4 800	393	6 793	8 325	660	65 800	1 205	16 050	0.21	0.17	$\frac{1}{29}$	$\frac{1}{130}$	$\frac{1}{60}$	$\frac{1}{20}$
C—6-ft., No. 3...........	0.187	0.173	4 800	423	6 823	4 200	590	59 800	988	......	0.07	0.11	$\frac{1}{27}$	$\frac{1}{200}$	....	$\frac{1}{20}$
6-ft., No. 1...........	0.187	0.173	1 375	400	2 235	4 010	240	42 700	759	......	0.01	0.06	$\frac{1}{26}$	$\frac{1}{85}$	....	$\frac{1}{8}$
6-ft., No. 2...........	0.187	0.173	1 585	486	2 561	9 500	476	43 300	695	......	0.04	0.11	$\frac{1}{12}$	$\frac{1}{87}$	....	$\frac{1}{8}$
6-ft., No. 3...........	0.187	0.173	1 855	401	2 876	5 500	280	54 300	975	12 000	0.11	0.04	$\frac{1}{28}$	$\frac{1}{79}$	$\frac{1}{38}$	$\frac{1}{8}$

TABLE 16.—MODULUS OF ELASTICITY BY FLEXURE OF PLAIN SLABS.

W', Dead load, in pounds.	W, Concentrations, in pounds.	Deflection, in inches.	Bending moment, in inch-pounds.	K, in pounds per square inch.	E, in pounds per square inch.
No. 1.—Moment of Inertia = 151.8 in.^{4} Section Modulus = 69.5 in.^{3} K = Extreme Fiber Stress.					
360	1 100	0.010	13 700	197	3 480 000
360	1 200	0.012	14 700	212	3 060 000
360	1 300	0.015	15 700	226	2 680 000
360	1 400	0.019	16 700	240	2 210 000
360	1 500	.....	17 700	254	....
No. 2.—Moment of Inertia = 140.8 in.^{4} Section Modulus = 66.1 in.^{3}					
350	1 100	0.0085	13 600	206	4 360 000
350	1 200	0.0115	14 600	221	3 450 000
350	1 300	0.014	15 600	236	3 020 000
350	1 400	0.0175	16 600	251	2 580 000
350	1 500	0.0275	17 600	266	1 740 000
350	1 510	......	17 700	268	....

TABLE 17.—DESIGN TABLE FOR PERCENTAGE OF REINFORCEMENT FROM 0.1 TO 1.0 PER CENT.

b = Breadth of slab;

d = Depth to steel;

k = Ratio of depth of neutral axis to depth of steel;

j = Ratio of effective depth to depth of steel;

M_R^c = Resisting moment of section, based on concrete stress;

M_R^s = Resisting moment of section, based on steel stress;

$M_R^c = (150\ k\ j)\ bd^2 \ldots\ldots\ldots\ldots\ldots\ldots = K_c\ bd^2$;

$M_R^s = (16\,000\ j\ p)\ bd^2 \ldots\ldots\ldots\ldots\ldots\ldots = K_s\ bd^2$.

Percentage.	k	j	K_c	K_s
0.1	0.22	0.93	31	15
0.15	0.26	0.91	35	22
0.18	0.28	0.91	38	26
0.19	0.29	0.90	39	27
0.20	0.29	0.90	39	29
0.21	0.30	0.90	41	30
0.22	0.30	0.90	41	32
0.24	0.31	0.90	42	35
0.25	0.32	0.89	43	36
0.26	0.32	0.89	43	37
0.27	0.33	0.89	44	38
0.28	0.33	0.89	44	40
0.30	0.34	0.89	45	43
0.32	0.35	0.88	45	45
0.33	0.35	0.88	45	47
0.35	0.36	0.88	48	49
0.36	0.36	0.88	48	51
0.38	0.37	0.88	49	53
0.40	0.38	0.87	50	56
0.5	0.42	0.86	55	69
0.6	0.45	0.85	57	82
0.7	0.47	0.84	59	94
0.8	0.49	0.84	62	108
0.9	0.51	0.83	64	119
1.0	0.53	0.82	65	131

DISCUSSION

Mr. K. M. Boorman.

K. M. Boorman,* Esq. (by letter).—The authors' conclusions as to fire resistance furnish added proof of the inadequacy of existing data on the proper thickness of fire protection of steel. The practice of specifying the placing of the reinforcement 1 in. above the exposed surface is based on an assumption which is not borne out by these investigations and others. A thickness of 2 in. is required as a protective coating for columns, and 1½ in. for beams. It is possible for such steel members, due to their comparatively large mass, to convey heat from their exposed surfaces quite readily. On the other hand, 1 in. only is required for the protection of slab reinforcement. This material, due to its small cross-section is, of course, more readily and to a greater extent affected by high temperatures. The specified 1 in. appears again inadequate when it is considered that in actual construction it is not possible, under economical working conditions, to insure this minimum thickness, and it is found to be reduced to ½ in. or less, and sometimes to nothing, that is, there is contact of the steel with the forms.

The writer is informed that the authors intend to extend their investigations, with a view to making recommendations as to the control of the quality of anthracite cinders. It might prove of value to determine the clinker-forming properties of different kinds and grades of coal, with a view of recommending a limitation of the use of cinders for concrete to the product of certain coals.

Mr. T. H. Boorman.

T. Hugh Boorman,* Esq.—This paper is of particular interest to the Engineering Profession as being in the line of more economical construction, which undoubtedly will be one of the principal features in the work of the Engineering Foundation. The utilization of waste material is of great importance. In America, with its immense natural resources, there has hardly been sufficient research on this question. The speaker, in his time, has seen the waste residua of gas-works and furnaces changed from useless and encumbering wastes to materials of commercial value, and hopes to see further developments with such an object in view.

In discussing the Report of the Special Committee on Materials for Road Construction, recently, the speaker had occasion to refer to Mr. Perrine's tests of one of the former waste materials, slag, in its connection with concrete construction, and would now suggest that he make tests and report on the utilization of another species of cinder, namely, the clinkers from garbage destructors or crematories.

In 1902 the speaker's attention was first attracted to the use of clinkers in concrete, in the City of Bristol, England, where they were

* New York City.

utilized by Col. Yabbicum in the construction of cement sidewalks. On returning to New York the speaker reported to Mayor Low the advisability of building a destructor, and, after a lapse of some years, one was erected on Staten Island, in furtherance of that suggestion, but without consultation with the speaker. In 1911, in investigating the municipal plants of a number of English cities, the speaker again found how extensively these clinkers were utilized in concrete construction, and, in addition, found that they were used by the Borough of Kensington, London, England, in the manufacture of asphalt blocks. On an inspection of the crematory at Atlanta, Ga., in November, 1914, it was found that, through lack of appropriations, the clinkers could not be utilized as suggested, as old tin cans and other metal refuse were all baked together, and the resultant product could only be disposed of in filling land. The speaker has had no opportunity to examine the several destructors erected in the West, and therefore does not know whether their output could be utilized in the same way as in English cities. Mr. T. H. Boorman.

A bill is now before the Governor of New York State for signature, authorizing the Borough of Manhattan to make a contract for the erection of a destructor to be operated by contractors. Such a project should be carried on by the municipality itself, for, though the speaker is not prepared to endorse a statement made that the receipts from the handling and disposal of garbage by scientific methods would pay the expenses of the Street Cleaning Department, he believes that a large revenue could be obtained therefrom and be devoted to paying such expenses. In view of this statement, it is hoped that the authors of this paper will include in their closing discussion a statement as to the strength of concrete construction with clinkers, in comparison with other aggregates.

Should there be any difficulty in obtaining samples of properly coked clinkers in America, the speaker would be pleased to furnish samples from some of the English destructors.

MYRON S. FALK,* M. AM. SOC. C. E.—The authors deserve much credit for the very valuable experiments they have made on the physical properties of cinder concrete. They have again demonstrated how extremely variable those properties are, and, in spite of these facts, the experiments show how safe the use of this unreliable material has been in the City of New York, where the construction of such beams in the field is far from realizing the conditions obtainable in the laboratory. The speaker has seen cinder concrete laid in freezing weather, subjected to alternate thawing and freezing, and he has seen it laid in driving rain, continuous for two or three days after laying, and yet in none of these cases has he seen failure. Mr. Falk.

* New York City.

Mr. Falk. The speaker doubts whether reinforced cinder-concrete beams, as ordinarily used for building purposes in the City of New York, are subject to any rational theoretical design. That the basic ideas on this subject are indefinite may perhaps be best noted by the fact that, in this discussion, various speakers have used indiscriminately the words "arches", "slabs", and "beams". As a matter of fact, practice shows actual conditions to be more complicated than any one has yet suggested, as the exact status may be described as a combination of arch, beam, and slab supported at four edges; and, further, in actual field practice, the load is never placed directly on the beam itself, as shown in the tests described by the authors, but is distributed on the beam (when wooden floors are used) by longitudinal sleepers covered with one or two thicknesses of flooring, or by a plain concrete slab, not reinforced, when cement-finished floors are used.

The speaker has no doubt that in many cases loads in buildings are carried to the steel frame without any stresses being developed in the cinder-concrete beams.

The Building Department of the City of New York acknowledges that, in certain cases of floor construction, arch action exists, and that tie-rods are required between the floor-beams to take care of the thrust. In other cases, it acknowledges that there is no arch action, or rather that the form of construction is safe without considering arch action, by permitting tie-rods to be omitted. In other words, cinder-concrete floor construction, as practiced in New York City for ordinary spans of from 5 to 8 ft., is not subject to rigorous design; this fact is emphasized when it is attempted to determine by theory the stress conditions in the steel reinforcement of test beams; as stated by the authors, impossible stresses of 300 000 lb. per sq. in. might be found to exist in the steel.

The speaker, in his practice, has had much difficulty in determining the reasons for the appearance of slight cracks in the ceilings of buildings containing floor construction designated by the authors as "flat-ceiling" or "bottom-flange construction". The usual attempt to explain these cracks is to say, offhand, that they are caused by expansion or contraction, or that the small quantity of concrete on the lower flange of the supporting steel beam causes the trouble. These explanations do not reach the seat of the trouble, and it would be advisable to have a thorough investigation to see whether the faultiness in this kind of construction cannot be eliminated.

Mr. Waite. GUY B. WAITE,* M. AM. SOC. C. E.—Accurate data, based on tests of commercial cinder concrete, have been needed for many years, and this paper is undoubtedly one of the most important contributions to this field of engineering. The Engineering Profession should feel

* New York City.

indebted to each and every one who has given time, talent, and money to produce these results. Mr. Waite.

The past and present manner of using cinder concrete in and about New York City, based on load tests, was good enough in the "old days", before any kind of reinforced concrete had been standardized. Since about 1900, reinforced stone concrete has been designed almost exclusively in accordance with the principles of the common theory of flexure.

Many of the larger cities have also put cinder concrete floor slabs, between steel beams, on the same basis for design as the stone concrete, using, however, lower working stresses for the former.

In their deductions, the authors recommend 30 as the ratio of the moduli of elasticity of cinder concrete and mild steel, and 300 lb. per sq. in. as the extreme fiber stress for the cinder concrete in the slab.

The authors, therefore, have substantially confirmed the unit stresses for cinder concrete which were adopted some years ago by Philadelphia, Baltimore, Boston, and Chicago, for the design of cinder concrete floor slabs.*

Although all cities have adopted definite unit stresses for the design of stone concrete, many, including New York City, do not recognize any unit stresses for cinder concrete.

The authors have pointed out the variableness of cinder concrete, which is very much greater than that of stone concrete. As cinder concrete is less reliable than stone concrete, it would seem that its design according to some definite standard would be more important than for stone concrete.

The approvals for the cinder concrete constructions now in use were based on tests of uniformly distributed loading over a sample slab. This sample was constructed as carefully as skill could make it; and the loading was placed so as to cause the least strain on the slab. Within the last few years, since Mr. Strehan has had charge of the testing for the City of New York, concentrated loading has been substituted for the former distributed loading, but, unfortunately, approvals from such loading gave so much lower results that those from the former tests are still the only ones in general use.

Before the change in the manner of testing, the system had developed into an art of competing to see who could go the farthest with the thinnest slabs, having the maximum spans, and with the greatest loads. So great became the proficiency that, toward the latter days, slabs with 3 in. effective depth and 8-ft. spans were approved for 250 lb. per sq. ft. The natural result of this form of testing was to surround cinder concrete construction with a kind of mystery, not existing in stone concrete of similar form.

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 1785.

Mr. Waite.

The particular form of reinforcement used in the mysterious construction was supposed to possess the magic power necessary to overcome the action of gravity and make the principles of mechanics, which act on stone concrete, inoperative in the case of cinder concrete. For several years the commercial interests which have been benefited by this chaotic condition have resorted to all kinds of methods to have a new Building Code passed which would arbitrarily specify cinder concrete in such a manner that their particular kind of reinforcement would have preference over all others.

Generally, stone concrete slabs, under the same conditions as the cinder, are designed, and their constructions superintended, by engineers; but, in the case of cinder concrete used in accordance with the original test, nothing is required except a promise to construct in a manner similar to the tested construction. The result is that Mr. Tom, Mr. Dick, and Mr. Harry, who have no knowledge of engineering, are doing most of this kind of work.

The tested slabs for these approvals were made with restrained ends, but those in buildings may be almost anything.

The authors have chosen for their tests ideally restrained and ideally free ends, the two extreme kinds of slabs which are the antitheses of one another. There was only one slab tested, called "special", which was between these two extremes. Therefore, the present investigation supplies no data tending to throw new light on the conditions in buildings where ends are only partly restrained or only partly free. In the average building about 25% of the floor area consists of outside slabs, next to walls or openings in floors.

This series of tests contains data concerning slabs made continuous (similar to construction tested and approved by loading) by stretching wire mesh over the supports and putting it under tension by stapling it down to the wood forms; moreover, in these tests, the concrete is carried 1 in. above the tops of the steel beams, thus entirely encasing the reinforcing wire mesh.

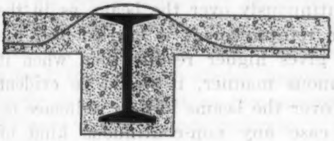
This form of construction, however, is not what goes into the floors of buildings. In most floors, electric and other conduits and floor sleepers are run crosswise of the steel beams, resting directly on the top flanges. This is partly for the convenience of having substantial level surfaces on which to lay conduits, and partly because, if raised higher, head-room would be taken from the floor.

The actual process of laying the wire mesh reinforcement in buildings is to roll it across the entire floor, allowing it to spread loosely over the tops of the steel beams. The speaker has never seen a single case where the reinforcement was brought tight over the flanges of the steel beams by stapling it down to the wood forms. The reinforcement is further loosened by the tamping of concrete. Thus, it is left loose and exposed over the flanges, instead of being embedded

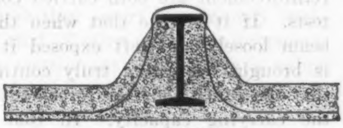
in concrete, as in the tests. Therefore, in practice, the slabs are only restrained to the extent of the initial tension in the reinforcement. Mr. Waite.

If there is play in the reinforcement over the beams, the end restraint will take place only when the slab has failed sufficiently to bring the end reinforcement into tension.

Further, in the bottom slab construction, a part of the slab is entirely below the steel beams, and the reinforcement is more or less exposed. The wire mesh which straddles the steel beams is often trampled up against the web, to get it out of the way of the concrete. Figs. 45 and 46 show the top and the bottom types of construction as actually built in the floors of most buildings.



TOP SLAB,
CINDER CONCRETE BETWEEN
STEEL BEAMS
FIG. 45.



BOTTOM SLAB,
CINDER CONCRETE
BETWEEN STEEL BEAMS
FIG. 46.

Some years ago the speaker proposed to use a continuous reinforcement over the tops of steel beams, for similar building construction, but, after trying it on one job, he gave it up as impractical, on account of the objections referred to. Therefore, his opinion is that the restrained slabs tested by the authors do not represent the conditions in the average building.

The free-end slabs were tested by the authors with rollers at the points of support, a condition not found in actual floors. The data obtained are too severe in the case of the free-end slabs and not severe enough in the case of the so-called fixed-end slabs.

If the intention of the authors is to start with ideal conditions and later work toward practical constructions, this should be made clear, so as not to confuse the uses for the deduced formulas, which are not applicable to present building constructions.

The authors find that one empirical formula is applicable to the free-end slab and an entirely different one to the restrained slab. Also, they find that the restrained slab is capable of carrying two and one-half times as much as the same construction with the ends free. If this is true, it must follow that, for conditions between restrained ends and free ends, such as is pointed out for both top and bottom slabs, in actual practice, formulas and results between these two extremes will result.

In testing the restrained slabs, the authors, in all cases, used wire mesh reinforcement. Why this was done is not known, unless it was,

Mr. Waite. as the authors state, that the wire mesh was furnished free. What little rod reinforcement was tested was used only in the free-end slabs.

Of course, the rods in the ideally constructed free-end slabs would appear poor when compared with the wire mesh reinforcement in the ideally constructed restrained slabs.

It seems strange that rods, so generally used in stone concrete slabs, were not used to some extent in the restrained slabs, thus affording a comparison of the two kinds of reinforcement.

It has been stated by one of the authors that the top slab construction, when brought down level with the top flanges of the beams, as shown in Fig. 45, will test higher than when the concrete and the reinforcement are both carried continuously over the beam, as in the tests. If it is true that when the reinforcement is carried over the beam loosely and left exposed it gives higher results than when it is brought over in a truly continuous manner, it would be evident that the continuous reinforcement over the beams has no influence on the carrying capacity. In that case any non-continuous kind of reinforcement will give equally good results. This, however, is denied by the authors when they state that their formula is "for slabs restrained between steel beams, with reinforcement thoroughly anchored, such as continuous mesh with proper cross-wires".

Therefore, there seems to be no escape from the fact that their formula for continuous reinforcement is not applicable to actual constructions.

It would seem as though data on the partly restrained construction, where one side of the slab is next to the walls, or next to the openings in floors, would have produced practical results. For some reason, only the two extreme types of slabs were tested, the rods being used on the free-end slabs. The result was that one type is two and one-half times as strong as the other.

The speaker would respectfully call attention to the experiments made at the University of Illinois, under the direction of Arthur N. Talbot, M. Am. Soc. C. E.,* in connection with the work of the Special Committee on Concrete and Reinforced Concrete. The purpose of the Committee was to gather data and suggest the proper manner of using concrete. These tests are used authoritatively in most recent treatises on reinforced concrete.

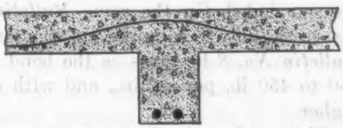
Inasmuch as rods, plain or deformed, are used almost exclusively for stone concrete (or any other concrete when in open competition), they were used also in these tests. After years of testing and after consideration on the part of the most talented engineers, the Committee, in its report,† recommends that for continuous slabs the bending

* *Bulletins Nos. 4, 8, 12, 14, 28,* and *29,* University of Illinois Engineering Experiment Station, issued during 1906 to 1908.

† *Transactions,* Am. Soc. C. E., Vol. LXXVII, p. 385.

Mr. Waite.

moment be taken at $\frac{wl}{12}$ and that for simple slabs (with free ends) it should be taken at $\frac{wl}{8}$. In other words, continuous stone concrete slabs, which are easy to make truly continuous, as seen by Fig. 47 (as there is no steel beam to obstruct the reinforcement), are only given a capacity one and one-half times that of the free-end slabs.



REINFORCED STONE CONCRETE

FIG. 47.

This differs somewhat from the results obtained by the authors on the cinder concrete, a material which they assert is extremely variable. In other words, they conclude that if wire mesh reinforcement is used, the variable, cinder concrete, continuous slabs attain a value nearly twice as great as recommended by the Special Committee for the more uniform stone concrete.

The authors seemingly go outside the data of this paper to call attention to assumed shortcomings of all kinds of rod or bar reinforcement. They state (page 581) that $350\,000\,000 \frac{da}{l^2}$ applies to slabs where continuous mesh with proper cross-wires is used; and that restrained slabs having longitudinal bars (of any kind) hooked over the beams are to be allowed but $250\,000\,000 \frac{da}{l^2}$. That is, bars (of any kind) hooked over the steel beams, they assume, are only two-thirds as strong as wire mesh. If the authors consider the wire stronger in tension than the rods, then the most that could be allowed in good practice would be from 16 000 to 20 000, or four-fifths instead of two-thirds.

They also mention that rods (of any kind) are deficient, but that wire mesh with sufficient cross-wires was found sufficient—though they do not explain what quantity of cross-wires is needed.

The speaker can find no data in the paper justifying such conclusions. He can find no data relating to actual tests of rods hooked over the steel beams as compared with tests of continuous wire mesh.

The authors claim that four slabs failed, due to the slipping of the rods in concrete. On these few samples they would base broad conclusions, and these without satisfying the speaker that there was actual failure in the manner assumed.

The speaker wishes to call attention to tests made by Professor Talbot, especially to those on the bond stress of rods in various mixtures of stone concrete, reported in the *Bulletins* mentioned previously. After extensive testing he concludes* that failure by slipping is difficult to detect because it is often the result of some other cause.

* *Bulletin No. 4*, University of Illinois Engineering Experiment Station.

Mr. Waite. He concluded (in the same *Bulletin*) that there was no failure in his tests, due to slipping, in the use of mild steel plain bars. In *Bulletin No. 8* he gives as the bond stress of plain rods of mild steel 350 to 450 lb. per sq. in., and with deformed bars a result somewhat higher.

The speaker also begs to call attention to tests on the bond stress of both plain and deformed bars in 1:2:4 cinder concrete at Stevens Institute.*

It would seem to the speaker that the authors are not justified in making conclusions from the results of only these four samples. In any event, rods which are hooked over the steel beams, or bent up in any way, have more resistance than mere bond to connect them to the slab. Further, in some deformed bars the projections are such as to offer an almost continuous resistance, independent of the bond of the bar proper.

Once more attention is called to the statement of the authors (page 581: "In the case of mesh, the cross-wires give sufficient mechanical bond."

For those who are not familiar with the construction of the wire mesh referred to (not explained by the authors), the speaker wishes to state that the triangular wire mesh is fencing wire, and that the cross-wires are not mechanically fastened to the tension wires, but are simply twisted around them. In the case of the electrically welded wire, the cross-wires are mechanically fastened, but occur, usually, only once in about every foot of length of the tension wires. It is to be understood that this wire mesh was not originally intended to be used as reinforcement in concrete, but to keep hogs and other animals from getting out of pastures. Notwithstanding the apparent insufficiency of the mechanical means to insure proper bond in the wire mesh, as compared with the deformed rods (which are especially made for the purpose), the authors gave a vast preference to the wire mesh.

The strong claims for using wire mesh instead of loose bars are made because the tension members in the wire mesh are spaced uniformly apart by the cross-wires. The importance of correct spacing is made superior to the importance of a sufficient quantity of reinforcement. Some engineers think the quantity of reinforcement is the more important. As a matter of fact, when a sufficient number of rods are used, even if not accurately spaced, the concrete is ample to distribute across from one tension member to the other. Further, there are numerous spacing devices which compel the accurate spacing of straight rods.

In the case of stone concrete slabs, in which engineers specify a definite sectional area of tension steel, bars of some kind are always

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 1787.

used, because there is no other form of reinforcement which can compete in price. The paradox in the competition of wire mesh is not because of its cheapness per pound, but because of the lack of pounds which are used, on account of its supposed magical powers. Mr. Waite.

The speaker has no commercial interests which he is trying to preserve by the discussion of this paper. He wishes to see the truth brought out and the fittest material survive. The peculiar interest taken by him is prompted by his experiences for the last 15 years in connection with investigations of new building codes for New York City. He believes that nearly every one of the many proposed building codes have been instigated by interests which hoped to be benefited thereby.

If the conclusions of the authors are allowed to be misunderstood, the writer can see how the paper may be used as justification of the present system of using cinder concrete in New York City, and how the one kind of reinforcement may be specified in the coming new code, which is now being prepared, to the exclusion of all others, as was attempted in the codes of 1912 and of 1913.

If cinder concrete is elevated to a plane where engineers use it according to units of strength, as has been done with stone concrete, then each engineer should be allowed his discretion as to what kind of reinforcement he thinks best. Certainly, in the minds of most engineers, these tests have not proved that wire mesh is superior to the forms of reinforcement generally used. If any one is thus convinced, then it can only apply to the two conditions tested. These two conditions are not those of the weakest part of actual floors.

Until all the actual conditions encountered in practice have been tested, conclusions and recommendations may be erroneous, and may lead to dangerous results.

The speaker thinks that at present it is safer to rely on the recommendations of the Special Committee on Concrete and Reinforced Concrete for the proper formulas by which to design cinder concrete, using 30 for the ratio of moduli and 300 for the extreme fiber, as recommended by the authors, than to trust to new formulas based on limited tests.

The formulas of the Special Committee are simple in form, generally understood, and undoubtedly express the actual conditions in buildings more closely than any others.

ALBERT OLIVER,* ESQ.—It is undoubtedly a fact that the tests, demonstrated so ably by the authors, go a long way toward deciding, in the minds of both advocates and skeptics, that cinder concrete fireproofing should be considered a safe and proper construction for buildings of any class, except possibly those in which the most extreme Mr. Oliver.

* New York City.

Mr. Oliver. conditions of loading and vibration obtain. However, there are many instances of the use of cinder concrete in much greater spans than those shown in the paper.

About ten years ago, the Singer Manufacturing Company erected several buildings of great dimensions at St. Johns, Que., Canada. Mr. Phillip Sidney, the Engineer of the Company, decided that reinforced cinder concrete should be used for all floor and roof slabs. There were three spans across the buildings, two outer ones of 19 ft. 10 in. from the wall to the center of the supporting beam, and an interior span of 16 ft. 10 in. from center to center of the supporting steel beams. The two outer spans rested on a 6-in. brick corbel at their edges, and were what is known as unrestrained slabs, that is, there was no continuity of the reinforcement beyond the outer edge of the slab. The reinforcement was 2 by 12-in. mesh, Nos. 3 and 10 Clinton electrically welded wire, to which was added ½-in. steel rods 12 in. apart. The concrete was a 1:2:4 mixture of cement, sand, and cinders, the outer slabs being 8 in. and the interior slab 7 in. in thickness.

As it was found impossible to get hard coal cinders, those from soft coal were obtained from the Canadian Pacific roundhouse at Farnham Station, and some were secured from Montreal. In no case were any hard coal cinders obtainable or used.

The speaker thought it was a crime to attempt such work, and so expressed himself. The engineer, however, had just completed a large quantity of work at South Bend, Ind., and had thorough confidence in the strength and other good qualities of cinder concrete. He felt himself well enough posted to be responsible, and did not court information on the matter. The work was done by the contractors with great care. The spans were very great, and the cinders were poor, but the cement and sand were of standard quality.

Rows of planers were placed on the 19 ft. 10-in. spans on each side of the buildings, the bays being 19 ft. 10 in. by 150 ft. On the 150-ft. stretch, however, the planers were set only far enough apart to permit of small hand-trucks being passed in for the removal of the planed cabinet work. The vibration, of course, is perceptible at all times when the machinery is in operation. In the other buildings there are great piles of castings and hardwood, and the speaker has always considered the load too great for the carrying capacity of these arches, yet, in all these years, not one crack or weakness of any nature whatsoever has developed in these floors.

The speaker, accompanied by R. D. Bradbury, Assoc. M. Am. Soc. C. E., visited these buildings about a year ago. On the day of this visit, a wheel pocket was being extended. To advance one of the machines into the room, it was necessary to cut a 10 by 36-in. slot into the cinder concrete arch. The Master Mechanic informed the

speaker that it had taken two men 14 hours to cut out the slot. This offered a chance to observe the condition of the reinforcement. Samples were taken and are now in the possession of Mr. Bradbury and the speaker. It was found that the wire was in first-class condition, that corrosion had not set in, that the galvanizing was still on the wire, and that there was no indication whatsoever of any deterioration. The slot was placed between two bars, so that it was not possible to observe the condition of the black steel reinforcement. Mr. Oliver.

The speaker believes that corrosion can only be prevented, especially where the reinforcement is not galvanized, by using a proper quantity of water in the concrete, and if it is difficult always to get the correct quantity, he believes it would be better to use too much water than not enough. The best results can be obtained by having the concrete so soft that to tamp it would be to splash it. When concrete for floor and roof arches will stand tamping, it is certain proof that it contains insufficient water. In getting into position, the first resistance met by a soft concrete which will flow is the reinforcement, and as it is certain that the cinders are not the first to flow past or around the reinforcement, it must be the finer part of the mixture and, therefore, the water and cement part must form the skin coating around the reinforcement. If water and cement are placed thoroughly around the reinforcement, corrosion will be almost eliminated. Any one who wishes a demonstration of this fact may readily obtain it by encasing a piece of steel in concrete, as described. After the concrete sets, break it away from the steel and begin to shave into that part of it which came in contact with the steel and one gets cement. The sand will not be far back, but, nevertheless, it is back of the actual coating, which will be cement.

The present use of cinder concrete for fire-proofing purposes is almost entirely the result of fire, water, and load tests on various systems, the term, system, being in most cases applied to the reinforcement used, for instance, the Roebling system, the Rapp system, the Clinton system, the Waite system, etc. Some tests have been very loosely conducted, but, in the poorest, there has not been, to the speaker's knowledge, a demonstration of the inability of cinder concrete to withstand the action of the fire test, as prescribed for New York City.

Undoubtedly, the best demonstration of the actual fire-proof qualities of cinder concrete is that given in the test house erected for Mr. Ira H. Woolson, at Norman and Monitor Streets, Long Island City. The successive fire and water tests to which its walls have been subjected prove beyond question the fact that a cinder concrete slab is less vulnerable to the action of fire and the application of water than any other building material now in use, not even excepting brick. The speaker has examined parts of the wall in this structure which

Mr. Oliver. have not been exposed to water, and has found that the grain of the lumber which formed the centering when the walls were being poured is still visible, thus proving that, where water is not applied to the heated surface, it is not possible to detect damage or deterioration in the concrete.

The old style of test load shown by the authors was that prescribed by the Bureau of Buildings, and that manufacturers and contractors made their tests accordingly was only to be expected. There is not much doubt that an arching effect occurred in many cases, but the speaker can show that in certain load tests made under Mr. Strehan's supervision, where the load was imposed on joists set on edge inside the line of the haunches, the joists being separated about 2 in. from each other, it was very difficult to trace an arching effect, and on these bearings loads were placed to such a height that Mr. Strehan asked for the discontinuance of the test, because it was regarded as dangerous to carry it further on account of the tendency of the load to slip and perhaps injure the workmen.

However, manufacturers and contractors may congratulate themselves that the new method of testing is now in use, and there does not seem to be the least reason for retracting the claims made heretofore that construction of this class will carry immense loads in a perfectly safe manner.

The speaker believes that the loading on third-points is the proper method, and is that used by J. R. Worcester and Rudolph P. Miller, Members, Am. Soc. C. E., Mr. Ira H. Woolson, Professor MacGregor, and the authors. Charles T. Main, M. Am. Soc. C. E., of Boston, Mass., A. W. French, M. Am. Soc. C. E., of the Worcester Polytechnic Institute, Messrs. Balcom and Darrow, and Warren and Wetmore, of New York City, all insist on center loads being applied, and tests made in both ways have failed to show any weakness in an ordinarily well constructed cinder concrete slab. Professor French insists on testing at the same time an outer and an inner arch, so that the effect on one in which the reinforcement is unrestrained may be compared with that on one having continuous reinforcement over the beams.

Fred W. Abbott, M. Am. Soc. C. E., lately conducted a load test on an isolated section of slab in the building being erected for Mr. P. A. B. Widener, of Philadelphia, Pa., Mr. Horace Trumbauer, Architect, and found, on a slab erected during the winter, a carrying capacity so far beyond the requirements of the Philadelphia Bureau of Buildings that he had every reason to conclude, with the concurrence of the officials of the Building Department, that reinforced cinder concrete formed a first-class construction.

There are matters affecting construction of this class which demand investigation by engineers. The matter of fire resistance has

been adequately proved and may be set aside as an accomplished fact. The matter of the strength of a soft mass of 1:2:5, or better still of 1:2:4 cement, sand, and cinders, with the steel reinforcement exactly placed as to centers and held in place while the concrete is being deposited, may also be put aside as an accomplished fact; but what can be said as to the absolute certainty of these matters when loose members are used for reinforcing and when their placing is delegated to careless, low-priced, uninterested labor, and this is exactly the class of labor engaged in placing the vital agent in reinforced concrete, both cinder and stone. The speaker has photographs which show the placing of a certain kind of loose members in the concrete of an important structure in New York City. No three members are equally spaced, and not a single member is on a line at right angles to the supporting flanges of the beam. The only straight pieces of steel running at right angles in the bays are the tie-rods. Mr. Oliver.

Now, the speaker asks, is this good practice, and does not such construction constitute a menace to life and property? He believes that the feature in this type of construction, which calls for the instant attention of the engineer, is not the fire-proof quality of cinder concrete—that has been proven—nor its load-carrying capacity when properly reinforced—that also has been proven—but the careless and improper spacing of reinforcement and the further abuse of actually leaving out many loose members where such reinforcement is specified; these are the particular points that call for drastic action by the Engineering Profession. The action must come from the engineer, whether he is employed by the city or in private practice. It must come from the Profession and not from the layman.

The speaker will offer to conduct a committee from the Society and prove every statement made as regards the placing of loose reinforcing members in concrete.

The authors have dealt with the matter of flaky soft substances in cinders. From experience, the speaker can say that from office buildings, hotels, gas and electric-light plants, the cinders gathered in New York City are of a high class. However, those obtained from certain congested points in the city, where more or less refuse is bound to be dumped, are not of good quality, and precaution should be taken at the building to reject any load of cinders showing more than an ordinary content of ashes or flaky materials, and any load wherein waste or refuse can be detected should be ordered off the premises.

When one considers that the area of cinder concrete floors and roofs already erected in New York City amounts to tens of millions of square feet, and that in no building where this construction has been used has a fire collapsed a single arch, and that not a serious break has occurred in any arch which has been in place for more than

Mr. Oliver. 30 days, it can be readily stated that to ask for the screening of the cinders, which form such an important part of this construction, would be to load up a first-class construction with an expense that would not be justified.

This matter has been fought out before the Code Committee of the Board of Aldermen. No more strenuous fight was ever conducted for and against any proposed change than that carried on by all interests concerned in the fire-proofing section of the proposed Code. During the past 10 years, the matter has been pretty well fought out, and it would seem that all are thoroughly satisfied that it should finally come up for disposition by Mr. Rudolph P. Miller.

Mr. Low. EMILE REED LOW,* ESQ. (by letter).—A test to determine the tensile strength of concrete made of granulated furnace slag, sand, and cement, as compared with cinder concrete, was made recently in connection with the construction of a pulp board mill, by the Beaver Companies, at Thorold, Ont. The specifications called for a 4-in. cinder concrete roof, mixed 1 : 2 : 4, and reinforced with No. 41 triangle mesh reinforcing wire, the concrete to be protected by an asphalted felt and gravel roofing. The final location of the new Welland Ship Canal had attracted numerous industries to this section, and these, together with the extensive work which the railroads were doing and the extensions being made to local factories, had placed a premium on cinders. In fact, during the middle of the summer, it looked as if it would be impossible to obtain cinders at any reasonable cost, and as pulverized furnace slag in large quantities could be secured cheaply, it was thought that slag might take the place of cinders.

A few years ago, the Carnegie Steel Company, of Pittsburgh, Pa., made a series of tests, extending over a period of 2 years, to determine the practicability of blast-furnace slag for use in concrete. The conclusions drawn from these tests showed that slag aggregate compared favorably with other materials, and that slag products have been used successfully in practice for several years. The concrete samples, however, were subjected to a compression test only. It was decided, therefore, to make a test for tensile strength and one that would follow more closely the special conditions.

Four concrete slabs were made up for testing, pulverized slag being used as the aggregate in two, and cinders in two. The slabs were made in wooden forms, 2 ft. 6 in. wide, 11 ft. long, and 4 in. high, and were reinforced with No. 41 triangle mesh reinforcing wire, manufactured by the American Steel and Wire Company. In all cases, the concrete was mixed in the proportion of 1 : 2 : 4 and was allowed to set for 7 days before being removed from the forms. The concrete was mixed in a "Wettlaufer" mixer to a consistency wet enough to

* Buffalo, N. Y.

run easily into the forms, and was not tamped as is usually the case in laying concrete roofs subjected to the great quantity of moisture present in paper and pulp mills. Mr. Low.

Cement.—The cement was made by the Canada Cement Company, of Montreal, at the plant at Port Colburne, Ont., and two samples were tested, with the results shown in Table 18.

TABLE 18.

	Sample No. 1.	Sample No. 2.
Specific gravity	3.062	3.12
Water for normal consistency	24.20%	24.00%
Initial set	2 hours 15 min.	1 hour 15 min.
Final set	3 hours 30 min.	2 hours 30 min.
Fineness, 100-sieve	97.05%	97.1%
Fineness, 200-sieve	79.82%	83.7%
	Tensile strength, in pounds per square inch.	
1-day, neat	330	325
7-day, neat	575	575
7-day, 1 : 3 sand	125	125
28-day, neat	620	615
28-day, 1 : 3 sand	300	215

Sand.—The sand was secured from the pits of the Clifton Sand and Gravel Company, near Niagara Falls, Ont. It has a reddish brown color, and from a distance seems to contain a large quantity of clay. The color, however, is due mostly to the presence of iron oxide, and although a close examination shows the presence of some clay and loam, the sand has been used locally for several years, and has made excellent concrete.

Slag.—The slag was secured from the Rogers-Brown Company, of Buffalo, N. Y. It is granulated by being run, in its molten state, into a pond of water. According to the statement made by this Company, the slag "averages about 40 to 50% lime, about 30 to 33½% silica, and about 16 to 18% alumina". The slag weighed 46 lb. per cu. ft., and ranged in size from ½ to 2½ in. The three elements contained in the slag are the essential elements of Portland cement, and, in a sample of this substance of good quality, average as follows: Lime, 62%; silica, 22%; alumina, 7.5 per cent. Although the slag aggregate contained the principal elements of Portland cement, and by pulverizing it, might be used as a cement, it would be low in strength and quick setting, due to the low percentage of lime and the high percentage of alumina. On the other hand, the high percentage of silica would tend to make a slow-setting cement which would not attain its full strength for a considerable time. By being granulated, however, rather than pulverized, it is not probable that the slag aggregate had any tendency to act as a cement.

Mr. Low.

Cinders.—The cinders were obtained from the Grand Trunk Railway, and were fair samples of the bituminous coal cinders made by the locomotives of this railway. Samples of the cinders averaged 45 lb. per cu. ft.

The slabs were tested by being weighed down with bags of cement which were placed symmetrically on them, beginning at the center and loading toward each end. The slabs were supported on 6 by 8-in. wooden beams, 10 ft. from center to center, this being the greatest center-to-center distance of the roof trusses in the building. The wooden beams were leveled and placed on solid foundations, so as to eliminate any chance of settlement.

The first test was made on one of the slag slabs when it was removed from the form, 7 days after the concrete had been mixed, and before it had dried out thoroughly. Nevertheless, the slab held up very well, and, as shown in Table 19, was able to support a load of 125 lb. per sq. ft. before breaking. The fracture occurred about 12 in. from one end; the other three slabs which were tested broke very close to the center. One noticeable feature of the tests was that, under the lighter loads, the deflections of the 7-day slag slab were exactly the same as those of the 21-day cinder slab; and although the latter, before breaking, supported a much greater weight than any of the others, its deflections for the respective loads were greater.

TABLE 19.—Deflections Under Various Loads.

Load, in pounds per square foot.	Slag, 7 days.	Slag, 14 days.	Cinder, 14 days.	Cinder, 21 days.	Remarks.
No load.	0	0	0	0	....
25	0.25	0.0625	0.0625	0.25	....
50	0.50	0.125	0.125	0.50	....
75	0.75	0.1875	0.1875	0.75	....
100	1.25	0.75	0.375	1.00	....
125	Broke.	0.9375	0.625	1.125	....
150	....	1.25	0.75	1.375	....
175	....	1.50	1.125	1.625	Load of 175 lb. cracked, 14-day cinder.
200	....	2.00	1.50	2.00	....
210	....	Broke.			....
225	....	....	1.75	2.25	Load of 225 lb. cracked, 21-day cinder.
235	....	....	Broke.	....	....
250	....	....	....	2.5625	....
270	....	....	....	Broke.	....

The second test was made on one of the cinder concrete slabs and the second slag slab, after they had set for 14 days, the first 7 days of which they remained in the forms, as stated previously. The tensile strength of these two compared very favorably, and although the ultimate breaking load of the cinder concrete was 25 lb. greater than

that of the slag, the former showed signs of fracture with a load of 175 lb. per sq. ft., when a small crack appeared near the center of the slab. The same feature was true of the 21-day cinder slab, which showed a slight fissure under a load of 225 lb. per sq. ft. On the other hand, the two slag slabs showed no signs of failure until immediately before their collapse; and yet their elasticity was nearly as great as that of the cinder concrete, as shown by the deflections of the different slabs, in Fig. 48. Mr. Low.

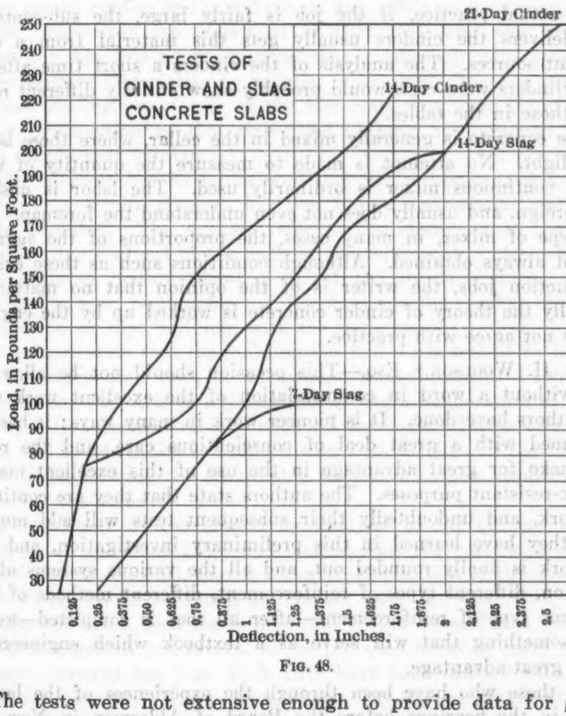


FIG. 48.

The tests were not extensive enough to provide data for general use, yet the results gave a very favorable comparison between cinder concrete and concrete with a slag aggregate. They also gave enough information to assure the engineers in charge of the work that sufficient strength would be secured for carrying the roof loads, whether cinder or slag aggregates were used. Before the work of laying the roof was begun, local conditions made it possible to secure cinders easily, and these were used in the concrete.

Mr. Diamant.

ARTHUR H. DIAMANT,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer cannot agree with Messrs. Perrine and Strehan that the formulas derived from tests, as mentioned in the paper, apply to working conditions obtained in actual practice.

Admitting that the cylinders were cast with cinder concrete mixed on the job, yet they were made by men familiar with constructing test cylinders. The test slabs also were made with cinders from the boiler room of Columbia University, carefully measured as to volume, and handled by men with more or less scientific training.

In actual practice, if the job is fairly large, the sub-contractor who delivers the cinders usually gets this material from a dozen different sources. The analysis of the cinders a short time after the test cylinders were cast, would probably show entirely different results from those in the tables.

The concrete is generally mixed in the cellar, where there is very little light. No attempt is made to measure the quantity of water, and a continuous mixer is ordinarily used. The labor is unskilled and foreign, and usually does not even understand the foreman. With this type of mixer, in many cases, the proportions of the aggregate are not always obtained. Although conditions such as these exist on construction jobs, the writer is of the opinion that no matter how carefully the theory of cinder concrete is worked up by the engineer, it does not agree with practice.

Mr. Woolson.

IRA H. WOOLSON,† ESQ.—This occasion should not be allowed to pass without a word in commendation of the excellent work which the authors have done. It is pioneer work in many ways; it has been performed with a great deal of conscientious care, and the results will make for great advantage in the use of this excellent material for fire-resistant purposes. The authors state that they are continuing the work, and undoubtedly their subsequent tests will add much to what they have learned in this preliminary investigation, and when the work is finally rounded out, and all the various systems of construction, different types of reinforcement, different methods of using the same type of reinforcement—after all that is completed—we will have something that will serve as a textbook which engineers can use to great advantage.

To those who have been through the experiences of the last ten years, in the hearings before the Board of Aldermen in New York City, in regard to the various types of fire-resistive material which should be accepted in Building Code regulations, this particular investigation is especially interesting. Those contests, as some know, were exceedingly hot. Days and weeks were spent in fighting over the

* Mongaup Valley, N. Y.

† Cons. Engr., The National Board of Fire Underwriters, New York City.

various types of construction, and cinder concrete was usually the center around which the battle waged. **Mr. Woolson.**

The speaker has been particularly interested in some of the results shown in this paper. The statement was freely and often made before the Board of Aldermen, that cinder concrete floors, when attacked by fire, would eventually burn out and drop of their own weight, because of the large quantity of unburnt coal which was contained in them. The speaker had occasion at that time to be in the midst of this squabble, and always contended that the quantity of coal found in the ordinary run of cinders had very little, if any, effect on its fire-resistive qualities. He made the statement at that time and was bitterly attacked for his one-sided opinion, that cinder concrete was the best fire-resistive material to be had; in other words, that its conductivity was the highest of any of the materials that were being used for floor construction at that time, and nothing has occurred since then to change that opinion.

Cinder concrete has its faults, and they are recognized; it has its weaknesses, in certain respects, but that it is a fire-resistive material, must be admitted. The authors refer to the circumstance that the speaker had dug coal out of the walls of the reinforced cinder concrete fire-test building, within about 1 in. of the inside surface. That is true, and is most remarkable. A few days ago the speaker repeated the inspection on that building, with the same result.

The authors state that the building has had a fire test of 28 hours; it has had more than that, at an average temperature of 1 700° for the whole time, and it is safe to say that it had a temperature of more than 2 000° for a considerable portion of that time. Pieces of bright unburnt coal could readily be found within 1 in. to 1½ in. of the surface of those walls, showing the remarkable resistance to heat conductivity possessed by the material.

The percentage of coal found by analysis in the cinders varies from about 12 to about 28%, and, considering the care with which the samples were collected, that is a fair representation of the quantity of coal which would be found in cinders used in New York City. The claim has often been made that coal should be excluded entirely from cinder concrete, and several of the proposed building codes which have been prepared for New York City have been drafted with that distinct requirement. These tests show that the requirement was entirely unjust, and that engineers must change their ideas on that matter.

In regard to the particular walls from which this coal was dug, it may be stated that the house was built for the speaker about seven years ago by a man who had never before built a reinforced concrete wall or structure of any kind. He was ordinarily intelligent, but had never had experience of that kind. The cinders were gathered from

Mr. Woolson. five different sources in Long Island City, and were not sifted; they were taken just as they came, and were made into a 1 : 2 : 5 mixture. The house was built without any supervision, other than that of the man who put it up, and it is giving splendid service in the work for which it was designed. It is good to-day for as many more tests as have already been made in it without any repairs except a little plastering where there has been local injury.

Another interesting thing in connection with that fire-test building is that the interior surfaces of the six chimneys which have taken up the products of combustion from all those fires, are to-day as smooth as they were when built. There is no pitting, and the little fins of concrete which formed between the boards of which the forms were constructed, stand there just as they did the day the chimneys were completed. This is another proof of the fire resistance of this material.

In regard to the question of design of cinder concrete floor arches, there will always be more or less difference of opinion. In time, it will doubtless be put on some reasonably acceptable engineering basis. The work done by the authors is of high grade, and deserving of our best commendation.

Mr. Worcester. J. R. WORCESTER,* M. AM. SOC. C. E. (by letter).—The information contained in this paper is valuable on account of the meagerness of data available with regard to this material. The results of the tests emphasize the wide variations which may be expected from cinders such as are commonly used in floor construction. There is little doubt that if a strictly vitreous clinker could be obtained, and all ash excluded, it would make a comparatively strong and reliable concrete; but the main object of using cinders would be lost, namely, the economy of the product. Cinders such as are readily obtainable will probably continue to include a large and widely varying proportion of fine, uncertain material, and the desideratum is a safe method of making use of the material commercially available.

In reference to this matter, it may be noted that the results of tests of screened and unscreened cinders reported in Appendix IV, which led to the conclusion that a loss of strength is to be expected from screening, are surprisingly at variance with what the writer would have predicted from a limited experience.

Other than this, the conclusions and recommendations of the paper seem to be in accord with previous investigations, and to be strengthened by the present study, though it is to be regretted that the authors see fit to give equal support to the empirical method of proportioning and that by the common theory of flexure. There does not seem to be any logical reason for adopting an empirical formula in which exponents are so ruthlessly manipulated and which must be

* Boston, Mass.

hedged around with limits beyond which they cannot safely be extended, when the theoretical treatment is so simple, and, comparatively, so satisfactory. Mr. Worcester.

In the theoretical method, it is doubtful whether it is wise to recommend for continuous slabs the moment coefficient of $\frac{1}{20}$ instead of $\frac{1}{12}$, even if the strength of these test slabs at failure does seem to warrant it. The objections are obvious. In the first place, it is known that the moment over the support exceeds $\frac{1}{20}$. Secondly, with imperfect continuity, which may be due to partial loading, flexible supports, or interruption of adjacent spans, the positive moment may actually exceed $\frac{1}{20}$. Then, again, the results of these tests may be somewhat misleading on account of arch action and partly restrained supports. There seems to be ample reason for a conservative value for this coefficient, and it would be better—if we should be ultimately convinced of the safety of decreasing our "factor of ignorance"—to do it by modifying our unit stresses rather than the flexure coefficients.

Gardner S. Williams,* M. Am. Soc. C. E.—In regard to the coal in cinder concrete, it may be suggested that, if air can be excluded, the carbon can be heated to a very high degree before any combustion occurs. Mr. Williams.

As to the methods of testing, where the sample is so large that a testing machine cannot be used, the speaker confronted that problem a number of years ago, and it was solved in a somewhat different manner from that described by the authors. As it may be useful in connection with this or some similar investigation, and might otherwise be overlooked, it will be briefly outlined.

The problem was to test a large girder of reinforced concrete composed of two beams and a deck. The deck was about 4 ft. wide. The span was a little more than 20 ft., and, in order to get the load on it, use was made of a series of pipes, about 3 in. in outside diameter and about 20 ft. long. The pipes were laid crosswise, one end projecting slightly more than the other beyond the edge of the girder, and the longer overhang was supported on spring balances hung from a scaffold or frame. The pipes were spaced carefully at equal distances, with short pieces of plank spanning them in pairs. The bags were piled on the planks in single piles, thereby preventing arching and distributing the load over the whole beam. The pipes, being able to roll and adjust themselves with the deflection, transmitted the load to the beam in a practically uniform manner. This method proved

* Ann Arbor, Mich.

Mr. Williams. to be very successful, and may easily be applied by any one who is interested in making tests on a large scale.

Of course, the extra length of pipe toward the spring balances made it possible to obtain the necessary load without having the pile of bags very high, and, in that way, without permitting the bags to come in contact with each other as the beam deflected. The work was carried on at the University of Michigan, under the supervision of Charles J. Tilden, M. Am. Soc. C. E.

Mr. Himmelwright. A. L. A. HIMMELWRIGHT,* M. AM. SOC. C. E. (by letter).—This paper reviews briefly the history of cinder concrete fire-proof construction and tests in New York City.

Having been identified with the early development and investigations of fire-proofing, and particularly cinder concrete floors, the writer has found much of interest in the paper. Many of the comments and criticisms of the authors in regard to the early investigations are justified and sound. In some cases, however, they have fallen into the same errors that proved to be stumbling blocks to some of the early investigators. As in the paper† by Guy B. Waite, M. Am. Soc. C. E., the investigation seems to have been conducted solely along the well-beaten path of dense concrete mixtures and conventional methods of reinforcement. The title, "Cinder Concrete Floor Construction Between Steel Beams" was broad enough to lead to the expectation that other forms of floor construction than the conventional reinforced slabs might be included; but in this respect the paper has proved disappointing.

Notwithstanding all that has been written on fire-proof floor construction during recent years, the writer is thoroughly convinced that a large field of this subject has been practically unexplored, and in this, sooner or later, will be found a more efficient and economical method of floor construction than any of the conventional methods now in use.

In reinforced concrete slabs, the metal reinforcement is placed on the tension side of the slab, and becomes more and more efficient in developing strength as it approaches the under surface of the slab and as its distance increases from the theoretical neutral axis. The nearer it approaches the under surface of the floor slab, however, the more it becomes exposed to heat in case of fire, and the sooner it becomes weakened by the effect of heat. This is a fundamental principle of weakness, which is present in every type of reinforced concrete construction.

Where a double construction (a floor and a separate ceiling with a space between) is desirable, as in office buildings, etc., there is no

* New York City.

† "Cinder Concrete Floors," *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 1773.

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particular advantage in having a horizontal surface on the under side of the floor. In the case of such buildings, therefore, a segmental arch type of floor could readily be used, and as this type is entirely independent of metal reinforcing elements, and the entire section of the arch material is in compression, it is a much superior fire-resisting type, and if cheap methods of centering and erection were developed, it would undoubtedly be a less expensive and more efficient type of flooring than any of the flat reinforced floor constructions now in general use. A segmental arch, of a uniform thickness of, say, 3 or 3½ in. of very porous material, would afford ample strength, and would be much lighter and more economical than the flat slabs ordinarily used. A porous concrete, such as is here suggested, weighs only about 7 lb. per sq. ft. per inch of thickness, and when erected on open centering, which does not retain the moisture, such a concrete can be made in any temperature without seriously impairing its strength or other qualities.

Conclusive tests by the writer, 18 years ago, and witnessed by a number of reputable engineers and builders, thoroughly established the fact that porous cinder concrete at a temperature of 38 to 40° can be placed on mesh or similar open centering in the usual manner in temperatures as low as 10° below zero without materially affecting the concrete. This is a commercial and practical advantage of enormous value in large cities where fire-proofing is frequently required to be built during the winter.

With the foregoing brief outline as to the possibilities of a porous concrete in segmental arch form, it is more or less astonishing that no progressive firm has given the attention and thought to this method of construction, which its commercial possibilities deserve.

Reverting now to the limited subject of the paper, the discussion will be treated under the headings into which the paper is divided.

Fire Resistance.—Considerable space is devoted to comment on what is termed the "pitting" on the surface of the concrete, which is stated to be "due chiefly to the combustion of small particles of exposed unburned coal, in addition to the effect of de-hydration". The statement is also made, that "one of the chief arguments, derogatory to cinder concrete as a fire-resisting material, is that unburned coal, occurring in the aggregate in appreciable quantities, when burned out, will produce flaws and honeycombing, and consequent loss in strength". Several illustrations are presented to show this "pitting" and "honeycombing" effect.

It would be interesting to know just how the quantity of unburned coal was determined in Table 5 giving the chemical constituents of cinders. The table states "carbon loss on ignition". If this was ascertained by heating to a high temperature, it is easy to account

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for the large percentage of "unburned coal" given in that table, running all the way from 11 to 28% of the total cinder.

Tests of the black particles which resemble unburned coal, made at the suggestion of the writer, showed that less than 7% of these particles would ignite at the temperature at which ordinary coal of the same size would be consumed and leave an ash residue. These tests proved that the black particles occurring in cinder are composed very largely of impurities, and their presence is thus explained, after passing through the furnaces of the steam boilers, etc., in which the coal was used.

It was also found, in the tests referred to, that, by raising the temperature, additional quantities of the black particles were consumed. It is well known, of course, that rock, slate, and similar foreign matter will disintegrate under the action of heat, and when exposed on the surface of the concrete to sudden cooling by water, will readily separate from the concrete and leave voids. It is the writer's contention, therefore, that only a part, and relatively a small part, of the pitting is due to unburned coal.

It would have been interesting if some tests or other data had been available to compare the fire resistance of dense or full concrete, with concrete of greater or less porosity. The writer's experience indicates that a full or dense concrete, in which practically all the voids are filled, is much more easily cracked and damaged by heat than a porous concrete; similarly, a stone concrete, with the same proportion of voids, cracks more readily on the application of heat than a cinder concrete.

In their behavior, the dense or full concretes resemble more nearly the different natural stones of which the aggregates are composed, in proportion as they are dense and full. In the case of cinder concrete, it is impossible to attain complete density, on account of the character of the cinder aggregate, which, in itself, contains considerable interior voids which cannot be filled by the cementing material, even when thoroughly tamped. It would be a comparatively simple matter to make tests of different degrees of porosity in cinder concrete, and show its relationship to fire and frost resistance. This would be valuable information in the development of more economic methods of construction. Lightness is highly important in economic designing, as it materially reduces the dead weight of the floors. This feature of concrete floor construction seems to have been wholly lost sight of in recent years. The reason of this neglect is undoubtedly due to incomplete investigations and the mistaken conclusions in regard to the corrosion of metal in cinder concrete.

Corrosion.—Under this heading is found the unwarranted recommendation of the Structural Association of San Francisco, which the writer condemned at the time it was made, shortly after the San

Francisco fire, as being entirely unjustified by the actual phenomena presented by that conflagration. Mr. Himmelwright.

In the past, and particularly in the case of work executed more than 8 or 10 years ago, reinforcing metal for floor construction was never protected against corrosion before being placed in the work. As a consequence, such metal was often oxidized over its entire surface, and was used in this condition. This was generally characteristic of Western practice, the writer having in mind the fire-proof floor work in San Francisco, Los Angeles, and other Western cities. In investigations made by the Structural Association of San Francisco, and by individuals, this fact was evidently not taken into consideration, and undoubtedly these are some of the "extreme and unexplained causes of corrosion" which have been pointed out from time to time.

In the examination made by the writer immediately after the San Francisco fire in 1906, very little of the exposed reinforcing metal was found to be free from corrosion, even where the concrete had been in contact with the metal. All these cases of oxidation were undoubtedly due to exposure to moisture in the salt air of the Pacific Coast before being placed in the work.

The conclusions of Professor Norton, of the Insurance Engineering Experiment Station, are also reviewed. These conclusions were discussed by the writer* some years ago, when they were first published, and the criticism made was that the investigations were not carried far enough, and that they were erroneously applied to structural steel.

These conclusions are based wholly on experiments with naked or unprotected metal embedded in the concrete. It is well known that structural steel, as furnished in practice, is usually protected with two or more coats of a suitable weather-proof paint, and this has an important bearing on the problem of corrosion, in so far as structural steel is concerned. Professor Norton's conclusions, and the writer's own investigations, have clearly proved that naked steel will be attacked opposite voids in the concrete, or where the cement is not in intimate contact with the metal surface. This fact, *per se,* widely published and advertised, has had the effect of restricting all effort, in the development of concrete floor construction, to the use of dense or full concretes, and, because of this unfortunate result, Professor Norton's findings, though correct as far as they go, have actually done more harm than good in the advancement of economic methods of cinder concrete floor construction. Had Professor Norton continued his investigations and ascertained that, in the absence of moisture, oxidation adjacent to voids progressed very slowly and could practically be neglected—which is a truth as important as the other—much of the harmful consequences of his conclusions would have been avoided.

* In *Engineering News,* and in other technical publications.

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When the writer first engaged in the business of putting in fire-proof floors, in 1896, cinder concrete was a new material; in fact, as far as he knows, his firm was the first to recognize its merits and use it for that purpose on a large scale. It was desirable to ascertain definitely whether or not it was suitable for fire-proof floor construction, and, more important than anything else, whether or not it had any harmful effect on the steel with which it came in contact, or any other undesirable properties for the purpose intended. As this involved a very thorough chemical investigation, the writer engaged Messrs. Booth, Garrett, and Blair, well-known chemists, of Philadelphia, Pa., to make the investigation as to the corrosive effects of cinder concrete. This investigation and report was made more than 18 years ago, and was very extensively distributed among the architects, engineers, and building contractors of New York City at that time. With the full information, as given by this report, and with all the information of nearly 20 years' experience available since that time, it seems incredible that the authors should make the statement that "the corrosion of steel embedded in cinder concrete is still a matter of conjecture". The detailed investigation of Messrs. Booth, Garrett, and Blair was so thorough and replete with information that the writer feels that it would not be amiss to reproduce it in its entirety, together with the supplementary tests and investigation suggested by the preliminary report.

"REPORT ON THE EFFECTS PRODUCED BY CINDER CONCRETE ON STRUCTURAL STEEL IN CONTACT WITH IT, OR EMBEDDED IN IT.

"March 3, 1898.

"MESSRS. JOHN A. ROEBLING'S SONS CO.,

117 Liberty Street, New York.

"GENTLEMEN.—Agreeably with the instructions received from Mr. Himmelwright of your company, we have had under consideration for some six weeks past the question stated in the heading, to wit, the effect of cinder concrete as used by you in floor arches, etc., upon steel embedded in it; and are now prepared to report on the matter as set forth in detail below:

"On January 7, we received from you, by express, samples, as follows:

"*Samples.*—1, Cement; 2, mortar sand; 3, anthracite cinder.

"We understand these samples to represent, respectively, one, Aalborg Portland cement; two, Cow Bay sand; and three, cinder from boilers of the New York Steam Co., using anthracite pea coal. We further understand that the proportions in which these ingredients are used in concrete are, one cement, two sand, and five cinder.

"*Corrosion of Iron and Steel.*—In dealing with corrosion of iron and steel, three active agents producing it are recognized. These are water, carbonic acid, and sulphuric acid. Pure water, free from

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carbonic acid, will not attack iron, but unprotected ironwork is rapidly corroded by either carbonic or sulphuric acid in aqueous solution or by contact with these acids in a damp atmosphere. It is, of course, a fact that in structural work the metal is generally protected by a covering of paint of some sort or other. Such coatings, however, vary greatly in the efficiency with which they protect the iron; many of them are destroyed themselves by the presence of either acids or alkalies, and all of them are likely to be removed in places by abrasion or other mechanical effect. Hence, for the purposes of this investigation, the coating on the iron may be dismissed as irrelevant, and the question then becomes, what corrosive effect a concrete of one cement, two sand, and five cinder, as per samples received, would exert on unprotected structural steel? Again, since both carbonic acid and moisture are always present in the atmosphere, it may be assumed that any corrosive effect which they can produce would be exerted in any case as conditions favored. It is not therefore necessary to make any investigation to discover the presence of carbonic acid; it is only necessary to consider whether it could produce corrosion on iron embedded in this concrete. The investigation thus narrows down at the outset to determining the presence of sulphides and sulphates in the concrete.

"There is shown by analysis of the samples received:

"Sulphates and Sulphides in Concrete.—

	Sulphuric Acid.	Sulphur as Sulphide.
"Cement	2.083 per cent.	Trace.
Sand	None.	Trace.
Cinder	0.213 per cent.	0.188 per cent.

"The figures for sulphuric acid and sulphur in the cinder are lower than we expected and lower than is generally supposed. On the supposition that the sulphur might be found in fine ashes instead of in the cinder, we requested you to send for another sample to be taken directly from the chute. Analyzing the fine ashes in this, we obtained:

"Sulphates and Sulphides in Ashes.—

"Sulphuric acid	0.161 per cent.
Sulphur as sulphides	0.338 " "

"This shows a little more. It occurred to us that the sprinkling of the ashes to cool them might dissolve out some of the sulphur, so we asked for a third sample to be taken directly from the ash pit.

"This analyzed:

"Sulphuric acid (SO_3)	0.144 per cent.
Sulphur as sulphides	0.116 " "

"This is the lowest determination of the lot. The conclusion we reach is that the sulphuric acid in the cinder may be reckoned at 0.20%, and the sulphur as sulphides at about the same figure—0.20 per cent. The sulphuric acid present in the samples is, of course, in combination—in the cement, as sulphate of lime, and in the cinder, probably also as the same salt.

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"The sulphides in the cinder are, however, capable of reacting with water to form sulphate, and if not combined with or surrounded by lime, are capable of corroding the iron.

"*Effect of Sulphuric Acid.*—The determination of the sulphuric acid present is important because of the remarkable effects produced upon the cement by this salt. These effects are two-fold, *viz.*, one, to modify the setting time; and two, to modify the tensile strength.

* * * * * *

"Without stopping to discuss the reasons of these phenomena (which are fully confirmed by numerous authorities), it is sufficient to point out that sulphate of lime in quite small quantities acts to retard the setting of cement and to increase its strength. In larger quantities, it is injurious, reducing the strength of the cement.

"*Total Sulphuric Acid.*—Reckoning the total sulphuric acid in the concrete to determine its proportion to the cement, we have:

"100 parts cement, sulphuric acid (say)		2.10 per cent.
200 " sand, " " (none)		
500 " cinder	(0.20 SO_3)	1.00 " "
	(0.20 S hydrated SO_3)	2.50 " "
		5.60 " "

or the total sulphuric acid present in the concrete would be 5.6% of the weight of the cement.

"*Fractional Part of Acid Only Effective.*—If this amount of sulphuric acid were actually able to react upon the cement, it would very seriously affect it; and, if we suppose the cinder to be ground to an extremely fine powder and intimately mixed with the cement before hydration, then some such effect would no doubt be produced. But as the sulphuric acid is actually disseminated through a mass of cinder in lumps, say ½ in. in diameter, many of which are hard and semi-vitrified, the conditions are quite unfavorable to chemical reaction, and a fractional part only of the acid can be supposed to be effective. That some of it is effective will appear below, but we have no hesitation in saying, one, that under the actual conditions this amount will not injuriously affect the cement; and two, that *per contra* if the cement can react to neutralize all this free acid, the iron embedded in the concrete will be fully protected from corrosion.

"*Test Specimen.*—To prove this by actual experiment, we made up very carefully a 2-in. cube of concrete, and three days after making it, a drip of distilled water was started over it, which was maintained for one week. An analysis of the solids dissolved out in this way showed that the proportions of lime and sulphuric acid were:

"CaO	13.28 per cent.
SO_3	13.80 " "

"The sulphuric acid present can combine with but 9.66 parts of lime, leaving 3.62 parts in excess of that required to neutralize the acid. There was no acid reaction, and the analysis shows that under favorable conditions there will be none.

"*Practical Test.*—For a practical test of steel embedded in a 1-to-2-to-5 concrete as used in New York, we made up from the samples furnished a concrete proportioned as follows: 540 grammes of cement; 1 080 grammes of sand; 2 700 grammes of cinder. Mr. Himmelwright.

"*Effect of Concrete on Naked Plate.*—This we packed in a box 6 by 8 by 5 in. in dimensions. In the center of this concrete, we placed a piece of thin and highly polished steel plate obtained by cutting and flattening out a piece of seamless bicycle tubing. After 28 days, the concrete was broken up and the plate removed. This plate we send you herewith. You will observe that the condition of its two sides are in marked contrast. On the bottom, the mortar was in contact with nearly the entire surface of the plate, and wherever it was thus in contact the metal is as bright as silver. On the top, where the mortar did not protect it, except in small spots, the surface of the plate is rusted and pitted by corrosion. This difference in condition of the two sides is due to the well-known fact that in tamping concrete the stone settles to the bottom and the mortar rises to the top. This is especially marked in meager concrete, in which the mortar is insufficient to fill the voids in the stones, which is the case in this mixture. In tamping the plate in position, the mortar has risen to cover its bottom, while in tamping the concrete from the top of the box, the cinder has settled down on top of the plate to the exclusion of the mortar.

"The top of this plate undoubtedly shows the corrosive effects of sulphuric acid and exhibits corrosive effects which, in a damp atmosphere, would be continuous to its complete ultimate destruction. The reason for this is that sulphuric acid or sulphates which will react with iron, act, in the presence of moisture, as continuous conveyors of oxygen to the metal. The acid first reacts upon the iron to form sulphate, and this, by a second reaction, becomes oxide, releasing the acid to again attack other particles of metal. In this way a small amount of free acid may, under favorable conditions, rapidly pit and destroy unprotected ironwork. This effect is exhibited in the rapid destruction of cast-iron pipes laid in cinder banks. The percentage of acid in the cinder, as the analyses given will show, is quite small, and the amount which is capable of chemical reaction, much smaller yet. But it is sufficient, by successive oxidation, to destroy the pipes rapidly. The same effect is seen in the rapid corrosion of ironwork from the sulphur fumes in smoke.

"*Conclusions.*—The inevitable conclusions to be drawn from these considerations are, one, that unprotected ironwork may be rapidly corroded by contact with this cinder in a damp atmosphere; but, two, that the iron will be completely protected from such attack if the cinders are embedded in a sufficient matrix of cement mortar.

"A 1-to-2-to-5 concrete, as now used, does not afford a sufficient matrix for the cinder, and unprotected iron in this concrete will corrode, and, in the presence of moisture, may corrode rapidly.

"*Theoretical Proportion for a Full Concrete.*—We suggest, then, that an amelioration of the concrete is desirable, increasing the relative proportions of the mortar. Thus, we have determined by experiment that the voids in the cinder are 54.5% of its bulk. These voids should be completely filled with mortar, allowing some excess of mortar besides, because its distribution can never be perfectly effected. The

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voids in the sand are 35% of its volume. Assuming, then, that the cement is to fill the voids in the sand and the sand the voids in the stone, the proportions of aggregates for a 'full' concrete of these materials would be, theoretically, 1 to 2.85 to 5.20. Allowing a little excess of mortar for practical work, as is necessary, this would give, for a working formula for full concrete, 1 cement, 2.75 sand, 4.75 cinder.

"*Formula Suggested.*—This formula we suggest for adoption in using this cinder concrete in your fire-proof-floor-arch system.

"This concrete should protect the iron from corrosion by either sulphuric or carbonic acid. The latter, it will be remembered, we dismissed at the outset of the discussion as present and able to act if conditions favored. In the presence of an impermeable concrete, as has been recommended, containing a considerable percentage of calcium hydrate, no rusting from carbonic acid and water is to be expected in your floor arch system. If it is impracticable to use such a mixture as has been suggested, under the building laws of the City of New York, we recommend, then, that a mortar of 1 cement to 2.75 sand be spread first in contact with the wire mesh of your arch system and with the surfaces of the beams. The cinder concrete, in proportion of 1 to 2 to 5, can then follow in usual course.

"We trust the above will fully cover the ground you desired us to investigate, and that this report will be satisfactory to you.

"Very respectfully submitted,

"Booth, Garrett & Blair."

"Messrs. Booth, Garrett & Blair,

"404-406 Locust Street, Philadelphia, Pa.

"Gentlemen.—We hereby acknowledge receipt of your special report on the effects produced by cinder concrete on iron embedded in it; also a piece of sheet steel tested. We are much pleased with the report, and we consider the amount of sulphuric acid and sulphides found in the cinder phenomenally small. We are a little surprised at the practical results which you have obtained in the sample of sheet steel sent us. It is evident that where the material is porous and the interstices not filled in solid with cement mortar, that the moisture in the concrete produces oxidation, while, on the other hand, where the cement mortar covers the entire surface of the iron, it serves as a very effective protecting material and prevents oxidation. These results complicate our problem considerably.

"If we should adopt a concrete such as you have suggested, using proportions of 1 of cement to 2.75 of sand and 4.75 of cinder, thereby securing a material which would have the interstices filled in with cementing material and be a solid, compact mass, we would have a concrete that would not resist violent changes in temperature. It has been shown very conclusively in fire tests that when a solid material like stone concrete, well rammed, so that the interstices are all filled with cement mortar, is heated to a glowing red heat and then suddenly cooled by the application of the fire stream, the material is unable to stand such a violent change in temperature and invariably cracks and disintegrates. The same is true as regards material of this character

when subjected to freezing. It would also be necessary in our system of fire-proof flooring to use a firm and water-tight centering, which would not only prevent the water from dripping out, but which would also make practicable considerable ramming of the material. The moisture would thus be retained in the concrete, and, if subjected to freezing weather, would freeze and crack and might be very seriously injured. **Mr. Himmelwright.**

"One of the claims which we now make is that our material, as we manipulate it and apply it, is porous and will sustain successfully the most violent changes of temperature without cracking. Now, according to your report, if we would use a material which will successfully protect the iron and prevent corrosion, we must modify it so that it will not give the best fire- and frost-resisting properties, besides making it very much heavier. In making a change of this kind, we would be losing many practical and economical advantages. On the other hand, if we continue to use the material as we do, there is the possibility of an initial oxidation, which, as we understand it, will last until the moisture disappears. It would, therefore, be interesting to know:

"1st. Whether or not this oxidation would continue in the absence of moisture, which condition would occur when the floors are thoroughly and completely dried and the building is under roof.

"2nd. Whether an initial coat of paint, such as is commercially applied to all ironwork, would not prevent oxidation until the moisture contained in the concrete disappears.

"We would be pleased to have you write us and express an opinion as to the two points above referred to, as well as any further suggestions that might occur to you.

"Very truly yours,

"JOHN A. ROEBLING'S SONS CO."

"Dictated by A. L. A. H."

"Philadelphia, March 11, 1898.

"MESSRS. JOHN A. ROEBLING'S SONS CO.,

"117-119 Liberty Street, New York.

"GENTLEMEN.—We are duly in receipt of your favor of the 5th, which has had our careful attention. We think the points which you have raised are very well taken, and fully appreciate their practical bearings.

"We have carefully considered the questions you raise, to wit:

"1st. Whether or not oxidation would continue in the absence of moisture, which condition would occur when the floors are thoroughly and completely dried and the building is under roof.

"2nd. Whether an initial coat of paint, such as is commercially applied to all ironwork, would not prevent oxidation until the moisture contained in the concrete disappears.

"And in replying to them in order, would say, first, evidently any corrosion of metal must go on very slowly in the dry atmosphere between the floor and the ceiling of a building. The pitting shown on the plate is, undoubtedly, due to acid attack, and this would probably slowly progress under these conditions; the rusting which comes from

Mr. Himmelwright. the water probably not; second, we are disposed to think, after carefully weighing the facts, that the iron would be protected by a good paint covering. In other words, we should hold that you could take this position, to wit, that if the manufacturer of the iron protected his material by a first-class quality of paint, you would guarantee that no corrosion would take place. We have under way a few simple experiments to cover these points and shall advise you of the results.

"We take it for granted that the painting of the iron work would not be within your province, and that therefore you could not decide what the paint should be. For your wire mesh and ribs, however, we would suggest that a paint of the asphaltum variety, free from oils, which can be dried up by acids or alkalies, would probably be better than an oil paint.

"Yours respectfully,

"BOOTH, GARRETT & BLAIR."

"Philadelphia, March 28, 1898.

"MESSRS. JOHN A. ROEBLING'S SONS CO.,

"117 Liberty Street, New York City.

"GENTLEMEN.—We beg to summarize the various reports heretofore made to you covering the question submitted to us for examination, to wit: 'Is any injurious effect to be expected from the use of cinder concrete in contact with steel beams in fire-proof floor arches?'

"We find:

"(A) 1. The sulphuric acid in the cinder will average 0.20 per cent. (two-tenths of one per cent.).

"2. The sulphur as sulphides in the cinder will average 0.20 per cent. (two-tenths of one per cent.).

"And that,

"(B) 1. These percentages of sulphuric acid and sulphur will not injuriously affect the cement.

"2. They will not injuriously affect good paint.

"3. They are sufficient to corrode unprotected ironwork more or less rapidly, depending on the presence or absence of moisture.

"We also find:

"(C) 1. That the mortar of the concrete, wherever it is in contact with the iron, fully protects it from oxidation.

"2. But that the proportions of 1 to 2 to 5 do not make a full concrete, and, hence, would not fully protect bare iron surfaces.

"It follows from this:

"(D) 1. That there would be some oxidation of bare iron or steel beams in this concrete.

"2. That in the ordinary dry atmosphere between floors of a fireproof building this oxidation could occur over a limited area, and would be extremely slow.

"3. That bare iron would be fully protected by the use of a 'full' concrete (say 1 to 2.75 to 4.75).

"4. That the iron is fully protected by good paint (red lead and linseed oil for example). Mr. Himmelwright.

"The conclusion we reach is that no injurious effects are to be anticipated from the use of this concrete above ground in the ordinary practice of construction now followed for steel frame buildings.

"Yours respectfully,

"BOOTH, GARRETT & BLAIR."

From this report, it will be seen that the preliminary investigation stopped practically at the point where Professor Norton's investigation ended. At the suggestion of the writer, a further investigation was made to determine additional and equally important facts. The correctness of the final conclusions of Messrs. Booth, Garrett and Blair have been fully borne out by the experience of the last 15 years.

It is apparent from the final conclusions that a porous cinder concrete will not cause the corrosion of structural steel, and a porous concrete, therefore, is a perfectly safe and economical material to use for fire-proof floors. This fact has been verified repeatedly in the razing of buildings, and in cutting openings in floors in numerous buildings where segmental arches of porous cinder concrete were used for the floor construction.

It is equally true, and experience has also confirmed this conclusion, that there is an initial oxidation in naked steel embedded in the concrete adjacent to all voids; but this oxidation progresses so slowly in the absence of moisture that it is practically negligible. Consequently, even in reinforced slab construction, a full or dense concrete, with its greater weight, though desirable, is not absolutely essential as long as the required bond stress and strength are obtained.

Another interesting field for investigation, with large promise of greater fire resistance without material loss in strength, is experimentation with reinforcing bars of such section as will obtain greater protection from the surrounding concrete. The writer refers to flat bars, wire trussing, strips of expanded metal, etc., set on edge and extending 2 in. or more into the concrete from the under side.

The writer made a number of fire and water tests with flat bars, 2 by $\frac{1}{8}$ in. and 2 by $\frac{3}{16}$ in., set on edge, with the under side of the bar $\frac{1}{2}$ in. above the under side of the slab. Very interesting and remarkable results were obtained with this type of reinforcement. The fire resistance of the construction was enormously increased and, strange to say, the strength, as determined by numerous tests, was approximately the same as that of the conventional section of bar, round or square, placed in the usual position of $\frac{3}{4}$ in. from the under side of the concrete slab. This result could only be accounted for by the fact that a large surface for bond stress was presented by a bar of this

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shape, and the additional fact that a flat bar set on edge had considerable value as a beam.

As a result of the writer's studies and investigations with this type of bar, it was adopted by the Roebling Construction Company, and, as the Roebling System "B", was installed in nearly 1 000 public buildings, offices, and hospitals throughout the United States during 1903 to 1908. The superior fire resistance of this method was thoroughly demonstrated in the San Francisco fire in the following manner:

There were, perhaps, 20 buildings occupied as offices and apartment houses, in which flat, reinforced-slab floor construction was used. Four or five of these buildings were fire-proofed by the Roebling System "B", and the rest were flat-slab floor construction with different methods of reinforcing, including meshes and rods of different varieties, but all placed near the under surface of the slab. In this class of buildings, the fire tests, as represented by the combustion of inflammable floor finish, trim, furniture, and furnishings, were only approximately $\frac{1}{2}$ to $\frac{3}{4}$ hour in duration. Temperatures from 1 800 to 2 200° Fahr. were obtained. This fire test, without the application of water, was sufficient to cause a considerable percentage of the floor arches in the ordinary methods of construction to sag under loads, such as safes, merchandise, etc., but such signs of weakness were not apparent in any of the buildings in which the "B" system of flooring, similarly loaded, was used. In some cases of other methods of flat-slab construction, as much as 30% of the entire floor area had sagged and had to be releveled or replaced when repairs were made.

The writer believes that it is possible to obtain largely increased fire resistance by these and similar methods of reinforcement, without any material sacrifice in strength or increase in cost, and that, in the near future, the advantage of superior fire resistance must receive the recognition which it deserves.

Mr. Hutchings.

JOHN B. HUTCHINGS, JR.,* ASSOC. M. AM. SOC. C. E. (by letter).—The experiments on cinder concrete, described by the authors, certainly represent a large expenditure of time and labor. The results are startling. Whether they will be substantiated later by other investigators is, of course, an open question. The authors have at least attacked an interesting and important question in architectural engineering with great vigor.

The firm with which the writer is connected has never made any tests on cinder concrete, and has used it only to a limited extent. As it is very difficult, if not impossible, to get sound, uniform, and clean cinders, the resultant construction must of necessity vary greatly, and, in using this material, it is well never to lose sight of that fact.

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* Louisville, Ky.

The writer agrees with the views of Guy B. Waite, M. Am. Soc. C. E., on the subject, as expressed in his discussion of this paper. It does not seem possible that any secret or patented method of placing the reinforcement in a slab of cinder concrete can give the resulting construction any magical or supernatural properties. Where his work has indicated the use of cinder concrete, the writer has followed his firm's experience in the use of stone and gravel concrete, designing by standard methods and merely selecting what appears to be a reasonable value for cinder concrete in compression. Mr. Hutchings.

The values used during the past two years are for a 1:2:4 mix, using commercially obtainable materials; they are:

$$f_c = 200 \text{ lb., and } f_s = 15\,000 \text{ lb.}$$

These values give the straight-line formula:

$$M = 22\ bh^2 \text{ (in inch-pounds),}$$

which is assumed to have a factor of safety of 4.

The necessary steel is then:

$$0.0014\ bh \text{ (in square inches).}$$

Numerous interesting and ingenious methods of placing reinforcing steel, and designs for such steel, are registered in the U. S. Patent Office; and, occasionally, in particular cases, it may be wise to make use of some of these patents. As a rule, however, it has been the experience of the writer's firm that work can be done just as economically and just as strongly with commercial material—with the added convenience of less worry from the misdirected enthusiasm of interested parties, and less chance of any "tie-up" in the work, due to delayed shipments from any cause.

As to the "spacing" of reinforcing steel, or securing it in the proper place, it may be stated that such work is one of the fundamental and inseparable operations in connection with the placing of reinforced concrete. It can be, and is, done successfully all over the world. The writer has had no particular difficulty in this matter, because his firm specifies "spacing-bars", "bar-seats", and "that all reinforcing steel must be inspected and accepted by the Architect or his representative before any concrete is put in place. Any concrete inadvertently placed before such inspection and acceptance shall be removed by the Contractor at his own expense". The idea is to know that the steel is properly placed and secured. It would seem to be just as logical to omit half the columns for a building, because it is very difficult to locate them correctly, as to omit 50% (or any part) of the calculated necessary steel in a slab, because of the difficulty of accurate spacing. The only guaranty of correct work in any branch of reinforced concrete is eternal vigilance.

Mr. Hutchings.

The Building Code of Louisville does not permit the use of cinder concrete for any structural member or for fire-proofing. The writer, however, sometimes uses it in roof slabs, and has never heard of the failure of a roof of this material, which had been properly constructed. The calculated loads on a roof, or floor, for that matter, are hardly ever realized, except in extreme cases, such as in dance halls, or during cyclones or fires. This fact no doubt has saved many faulty pieces of work.

As to the probable corrosion of steel embedded in cinder concrete: About 1895, Professor C. Richter and Professor J. M. White, of the Architectural Department, of the University of Illinois, as associate architects, designed and built the handsome $150 000 Romanesque Library for that University. The roof slabs were of cinder concrete, and 4 in. thick, with "diamond-mesh" expanded metal about ⅛ in. thick, and of 3 by 5-in. mesh, as the reinforcement. The roof construction was of steel with a covering of slate. On examination, the writer could find no evidence of corrosion due to the cinders. As another instance of this kind, the Columbia Building, Mr. C. A. Curtin, Architect, at Louisville, Ky., may be mentioned. This building is at least 20 years old, and was the first in the city to have beam and tile floors. The office of the writer's firm has been in this building about 18 years and, at various times, when repairs or alterations have been made, he has examined nails, pipes, anchors, and beams, which happened to be in contact with the cinder concrete fill (about 4 in.), placed between the hollow tile and the wood floor. As far as he is able to judge, no damage has resulted from the use of cinders. At present, it is believed that cinders are "safe" for use in dry locations, but with first-hand knowledge of these two examples only, the writer's firm is not ready to use it in its own practice without proper precautions. Its present specifications call for, and the firm sees that it gets, "a sub-filling at least 1 in. thick, of stone or gravel concrete, next to all iron conduits, beams, etc., where cinder concrete is to be placed". The firm has not used cinder concrete as "fire-proofing".

This paper is very acceptable, and, considering the interest of the problems and the large amount of money involved, the writer wonders why more papers on architectural subjects are not presented before the Society.

Messrs. Perrine and Strehan.

HAROLD PERRINE* and GEORGE E. STREHAN,† JUNIORS, AM. SOC. C. E. (by letter).—The title of this paper was worded so as to indicate the type of construction under discussion, and the object was to co-ordinate past and present practice with safe and sane design.

It was not desired to encroach on present accepted design of reinforced concrete, although tests of completed buildings of this type,

* Wilmington, Del.

† New York City.

such as the Turner-Carter Building, in Brooklyn, N. Y., show computed stresses under working loads considerably in excess of actual measured stresses under equal test loads. In the type of construction treated in the paper, the main structural units of the building, such as beams, girders, and columns, are of steel, and the cinder concrete, reinforced or plain, simply forms a filling between the beams on spans varying from 4 to 8 ft. **Messrs. Perrine and Strehan.**

The history of construction for a period of approximately twenty years, is well covered by this paper and that* by Guy B. Waite, M. Am. Soc. C. E. In the latter paper, however, an attempt was made to place floors of this type on the same basis as straight reinforced concrete construction, that is, to use the common theory of flexure.

After twenty years of satisfactory service in New York City—the only criticism of which was the illogical method of control—it is incongruous to find in a building code, adopted in January, 1915, for a city of approximately 80 000 population, under materials of construction prohibited, the items "cinders, plaster of Paris, sulphate of lime, and all similar injurious materials". The writers do not claim supernatural powers for any one type of reinforcement, such as that originally intended "to keep hogs and other animals from getting out of pastures". The many ramifications of methods of building floor fillings are being investigated in the order in which the extent of application would appear to warrant such study.

Mr. Falk believes that the floor filling, as ordinarily used for building purposes, is not subject to any rational theoretical design, but in order to overcome the many inconsistencies of present practice, which were adhered to in the proposed codes of 1912 and 1913, it is necessary to find a logical method of control. Mr. Worcester's proposal to increase allowed unit stresses instead of decreasing bending moment coefficients does not strike the writers as logical, inasmuch as the ultimate resistance of the material to internal stresses is known, and, therefore, safe working stresses with reasonable factors of safety follow at once. The magnitude of such internal stresses, however, is unknown. The writers deemed the more reasonable method of approach to be a direct comparison of actual load-carrying capacity between the construction variously described as an arch, beam, or slab, supported at four edges, or a combination of all of these, and the simplest type of reinforced concrete beam, inasmuch as the static condition of the latter construction under external loads is a definite matter. Moreover, Mr. Worcester's method would necessitate the use of varying working stresses for the same material when used in different types of construction. It would seem that this method of analysis will place the construction on what Mr. Woolson terms a "reasonably acceptable engineering basis".

* "Cinder Concrete Floors", *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 1773.

Messrs. Perrine and Strehan.

Mr. Waite cites the unit stresses allowed in cinder concrete construction in other cities. Table 20 has been prepared showing the comparison between the loads which could be permitted in other cities with those approved in New York City in the past by means of tests.

In the proposed fire-proofing section of the New York City Code (1915), the complex character of this type of floor construction is not only recognized, but the "cloak of mystery" is also discarded, when stone concrete is used as a filling between steel beams on spans up to 8 ft. In this Code, the following formulas are proposed when cinder concrete is used:

For non-continuous bars or mesh.........$w = 14\,000 \dfrac{d\,a}{L^2}$

For hooked bars.........................$w = 18\,000 \dfrac{d\,a}{L^2}$

For continuous mesh...................$w = 26\,000 \dfrac{d\,a}{L^2}$

w = total load, in pounds per square foot;
d = depth to steel, in inches;
a = area of steel, in square inches per foot of width;
L = span, in feet.

When stone concrete is used, these coefficients are increased to 16 000, 20 000, and 30 000, respectively.

TABLE 20.—COMPARISON OF PERMISSIBLE LIVE LOADS, IN POUNDS PER SQUARE FOOT. (ASSUMED DEAD LOAD, 60 POUNDS PER SQUARE FOOT.)

Slab, 4 in. thick.

Reinforcement, continuous wire mesh.

					BENDING MOMENT = $\frac{1}{12}$ $W\,l$.			
Span, in feet.	Area of steel per foot of width, in square inches.	Approved in New York by test.	Proposed 1913 code.	Proposed on basis of unit stress, New York City	Philadelphia.	Chicago.	Baltimore.	Proposed empiric formula, New York City.
6	0.074	175	100	30	30	40	25	100
	0.087	200	150	45	40	40	35	130
	0.101	300	200	60	50	50	50	160
	0.119	400	250	75	55	50	70	200
7	0.087	165	100	35	35	45	25	130
	0.101	200	150	50	50	50	40	160
	0.119	300	200	65	60	60	60	200
	0.140	....	250	90	65	60	80	240
8	0.101	....	100	35	40	50	35	120
	0.119	120	150	55	55	60	50	150
	0.140	250	200	75	70	65	65	185
	0.163	....	250	95	70	70	90	230

Messrs. Perrine and Strehan

TABLE 21.—PERMISSIBLE UNIT STRESSES.

City.	Fiber stress in concrete, in pounds per square inch.	Ratio, *n*.	Mixture.	Steel stress, in pounds per square inch.
Philadelphia	250	30	1:2:4	16 000
Chicago	245	30	1:8	18 000
Baltimore	300	30	1:2:4	15 000
New York City (as proposed)	300	30	1:2:5	16 000

Mr. Waite also criticizes the fact that, in the tests cited in the paper, only cold-drawn wire-mesh reinforcement was used, and that apparently the coefficients for bars were based on assumption. Furthermore, he believes that the bottom-slab or flat-ceiling type of construction will give remarkably different results. The writers wish to state that the coefficients for the rod reinforcement were based on official Bureau of Building tests which were made under conditions identical with those carried out in the investigation, as shown by Fig. 5. It is furthermore proposed to make similar tests in a continuation of the present investigation.

During the present year (1915), the bottom-slab type of construction was tested with wire-mesh reinforcement, and a summary of the results is given in Table 22.

TABLE 22.—BOTTOM-SLAB CONSTRUCTION.

Slab No.	Span.	End condition.	Percentage of reinforcement.	Deflection, in inches.	Total uniform load, in pounds per square foot.
B. C. 1	7 ft. 1 in.	Fig. 49	0.26	6.13	313
B. C. 2	....	Fig. 49	....	4.75	563
B. C. 3	....	Fig. 49	....	7.5	695
B. F. 1	....	Fig. 50	....	2.8	288
B. F. 2	....	Fig. 50	....	3.4	309
B. F. 3	....	Fig. 50	....	3.8	329
F. 1	....	Fig. 51	....	1.45	106
F. 2	....	Fig. 51	....	4.6	126
F. 3	....	Fig. 51	....	5.15	111

In this case, again, the attempt was made to compare the load-carrying capacity of the filling between the steel beams with the simply supported slab. It will be noted that the bottom slab with continuous mesh is from two and one-half to four and one-half times as strong as the simply supported slab, whereas, the top construction in the 1913 and 1914 tests showed a ratio of 2½ to 1 for the continuous reinforced filling as compared with that for the simple slab. The bottom slab continuous is approximately one and three-quarter times as strong as the bottom slab non-continuous. Taking the bending

Messrs. Perrine and Strehan.

moment coefficients for the simply supported slab as $\frac{1}{8}$, the coefficients for the various types of filling are as follows:

Top construction, continuous .... $\frac{1}{20}$

Bottom construction, continuous .... $\frac{1}{20}$ to $\frac{1}{35}$

Bottom construction, non-continuous .... $\frac{1}{12}$ to $\frac{1}{20}$

In answer to Mr. Waite's criticism that the conditions assumed do not bear out the construction as installed in practice, it is only fair to say that continuous mesh is often carelessly placed, in that it is not laid in proper position, nor are all the kinks always removed. In an investigation of this nature, however, it is necessary to eliminate all uncertainties and to assume more or less ideal conditions in order to arrive at definite conclusions.

Mr. Diamant believes that the high results obtained are due to the fact that the slabs and cylinders were cast by men experienced in making test specimens. In the 1913 series, the slab and cylinder specimens were cast by students, from concrete taken directly from the contractor's batch at the site of building operations at Columbia University, as explained in the paper. The 1914 and 1915 slabs were made of concrete mixed and cast by students, and the cylinders of typical concrete were obtained from actual jobs throughout the city by students who were not thoroughly expert in making test specimens.

Mr. Himmelwright's criticism that nothing new had been developed, and his disappointment that more economical forms of construction were not treated of, can only be answered by a re-statement of the writers' object, "to place the present inconsistent practice on a logical basis". Exception, however, is taken to the argument for the use of reinforcement in the form of flat bars or beams of special shapes. This type of reinforcement, extending both above and below the neutral axis of the slab, has frequently failed in test by the shearing of the bars through the concrete fill.

As to the matter of screening the cinders or changing the mix to obtain the ideal dense concrete: this appears to the writers to be unnecessary, in view of the limited application of the material to a filling between steel beams. Mr. Oliver would recommend a 1:2:4 concrete, and Mr. Himmelwright, for a specific New York City cinder, shows that the proper working formula is $1:2\frac{3}{4}:4\frac{3}{4}$. Because of the variableness of the material, it is impossible to fix any one correct mixture; when ash predominates, the use of less sand is necessary; and if the cinder consists wholly of vitrified steam-coal clinkers, a large proportion of sand is necessary in the mixture. Fig. 11 shows the fracture of the typical cinder concretes. Cylinder A shows a rather poor cinder concrete containing a large percentage of dust and large clinkers, resulting in a porous concrete. Cylinder B_1 represents apparently good material, but was hand-mixed and rather porous.

Cylinders B_2 and C were made of good material, and represent good concretes. The samples, B_1, B_2, and C, represent cinders showing practically the same gradation of sizes; B_1, however, was mixed 1 : 1 : 5 and the others 1 : 2 : 5, as shown by Table 6. Messrs. Perrine and Streban.

The specimen, B_1, however, fell very low in strength. This cinder also contained the highest percentage of unburned coal (28.10%), and was hand-mixed. The cinders represented by Cylinder A show by

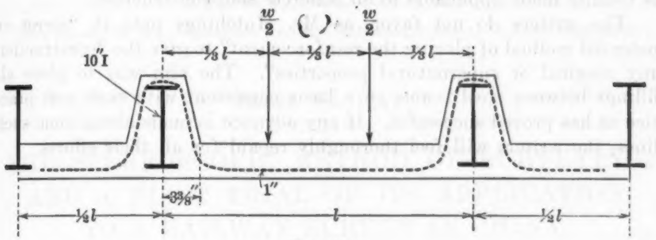


FIG. 49.

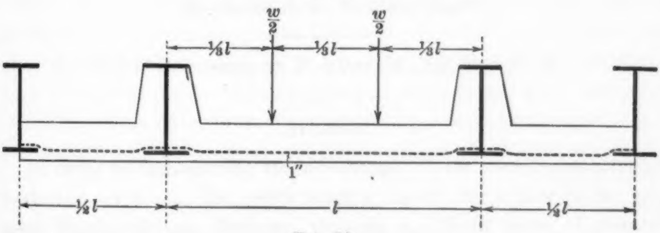


FIG. 50.

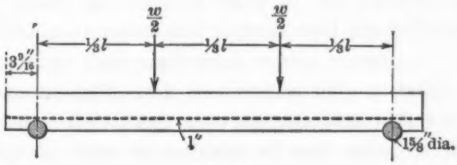


FIG. 51.

mechanical analysis the largest percentage of fine material, 13.7%, passing the 100 sieve. A comparison of the mechanical analyses in Tables 6 and 9 indicates that the aggregates for both slabs and cylinders are quite similar, although the strength of the slab concrete is 50% greater. This can only be accounted for by ordinary care in mixing, and would indicate the necessity of more supervision in the field.

Considerable criticism has arisen because of the use of the theorem of three moments, assuming fixed supports at the same level, constant

Messrs. Perrine and Strehan.

moment of inertia, and full continuity of construction. The theorem was used in this form with a full knowledge that it was not entirely applicable, but that, in view of the indeterminate character of many of the existing conditions, the influence exerted by the varying cross-section and non-fixity of support could be neglected. It was necessary, however, to have a basis for comparison of the simple and restrained types, and this special form of the theorem of three moments is usually made applicable to all concrete slab construction.

The writers do not favor, as Mr. Hutchings puts it, "secret or patented method of placing the reinforcement" to give the "construction any magical or supernatural properties". The aim was to place all fillings between steel beams on a basis consistent with such past practice as has proved successful. If any advance is made along some such lines, the writers will feel thoroughly repaid for all their efforts.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

TRANSACTIONS

Paper No. 1342

THE STEREOSCOPIC METHOD OF SURVEYING, AND A FIRST TRIAL OF ITS APPLICATION TO A RAILWAY SURVEY IN CHINA

By Georg A. G. Müller, Esq.*

With Discussion by F. Lavis, M. Am. Soc. C. E.

Synopsis.

In order to demonstrate the advantages of the stereo-photographic method of surveying, the writer made a survey, for a part of the proposed Hankow-Ichang Railway, through a difficult piece of country in the very rough and tortuous valley of the Yangtze Kiang River, in China. The instruments and methods used are described and illustrated in detail by their application to this survey.

The photo-theodolite is a combination of a metallic box camera and a theodolite; the optical bases are measured with a rod, mounted on a tripod; the three co-ordinates of each point on the negatives (or diapositives) are measured on the stereo-comparator, which, perhaps, is as important an instrument as the photo-theodolite in stereo-photographic work; the points are plotted on a special drawing device consisting of a glass plate to which steel rulers are pivoted and clamped. The other apparatus consists of double plate-holders, photographic plates, etc., etc.

* Regierungs Baumeister: Dipl. Ing. (Berlin); Professor of Civil Engineering to the Government University, Peking, China.

The general plan of the survey included a preliminary reconnaissance of the territory, a triangulation network with a base line in the open part of the valley, and a line of levels in the bottom of the valley establishing bench-marks from which the elevations of the photo-theodolite could be determined or checked. The photographic stations or standpoints were then selected, and from these the photographs were taken, either at the time or whenever the weather and the position of the sun were favorable.

On account of the newness and possible importance of stereophotographic surveying, the theory is illustrated and described in considerable detail, and also the manner in which the points on the photographs are measured and plotted.

It is claimed for this method that it is simple, about seven times as rapid as a tacheometrical survey, is not expensive, and is especially suitable for work in rough and difficult territory.

Although the paper relates only to the application of this method to a preliminary railway survey, it may be used for other purposes, for instance, stereographic pictures, taken from a fixed base line, will serve to record the progress of work, as they will enable one to check quantities of earthwork, masonry masses, dimensions, the agreement of structure with design, etc., and may be examined at headquarters at any time.

While making a topographic survey in Hanyang-Wuchang during the summer of 1913, with his students, the writer had an opportunity to demonstrate the stereo-photographic method of surveying to Councillor Linow, Chief Engineer of the Hankow-Ichang Railway. The particular advantages and the results obtained recently in Europe by this method convinced the Chief Engineer that it might be advantageous to make a trial of it in China, and the writer was commissioned to make a preliminary photographic survey of the most difficult part of the proposed line during the summer vacation.

The party consisted of the writer as Chief Engineer, and later two German engineers, six of his former students, now graduated, ten Hupeh students, graduated in Japan, and a few draftsmen.

The preliminary line, proposed after the first reconnaissance, runs for about 20 li* (7 miles) nearly parallel to the Yangtze Kiang,

* 1 li = about ⅓ mile.

thence for 20 to 30 li (7 to 10 miles) through hilly, rolling country, and, finally, after crossing a tributary of the Yangtze, reaches the district under discussion. The valley is extremely rough and tortuous, and, with its steep rocky bluffs and wooded slopes, presents great difficulties to the locating engineer, so that the work served as a valuable test of the possibilities of this method of surveying.

This valley, measuring 10 to 12 li (3.6 to 4.3 miles) on the line and about 17 li (6.1 miles) along the river, represents the *pièce de résistance* of the Hankow-Ichang Line. Probably it will be the most expensive part, on account of the many structures required. For this reason the survey was first commenced on the scale of 1 : 2 500; the rolling country was to be surveyed later.

General Plan of the Work.

The line of the reconnaissance was marked by flags on high bamboo poles every 400 to 600 m. (1 300 to 2 000 ft.). Then a triangulation net was made over the whole district to be surveyed, starting from a favorably located base line in the broad open part of the valley. It was endeavored to include in this triangulation as many points on the line as possible, and, in order to avoid confusion, such points were indicated by poles with flags at their tops; the poles at the triangulation points also bore flags in the middle. Each flag carried a wooden target at a height of about 1½ m. from the ground. Altogether, about 30 points were established.

A base line of about 214 m. (700 ft.) was carefully measured with 5-m. wooden measuring rods which had been standardized in Peking. From each point the direction of all visible points was determined within 10″ with four readings. At the same time, most of the elevation angles were taken.

The triangulation served a double purpose: First, it made possible the orientation of the standpoints of the photo-theodolite. Secondly, its points (the elevations of which were determined trigonometrically), formed an important control for the photographic plates.

As explained later, this check served, even when the bamboo poles did not appear on the plates. The targets answered as base points for the stations, as the lower parts of the poles were frequently concealed by grass or otherwise, the average height of the theodolite above the ground being 1½ m.

The "distances" of the triangulation (400 to 1 200 m.) (about 1 300 to 4 000 ft.) were calculated and plotted, and precise levels were run along the bottom of the valley. From these the elevations of the photo-theodolite were determined or checked.

The selection of the photographic standpoints was made, the directions and base lines for the stereographic exposures were taken, base lines were measured and oriented with reference to the triangulation, and occasionally a few elevation angles were measured by the photo-theodolite. The points for the camera were marked by pegs, and the photographs were taken, if the weather and position of the sun permitted, or postponed until a more favorable time. The plates were developed and diapositives and prints were made at headquarters.

The next stage of work was the taking of measurements from the plates in the stereo-comparator. This is a most important instrument and on it the progress of the survey depended. It was kept continuously at work during daylight, by shifts of operators, throughout the whole survey.

The determined co-ordinates from each pair of plates were plotted on tracing paper pasted on especially arranged glass plates, and the orienting directions were laid out. Then these sheets (one for each pair of photographic plates, Figs. 18 and 19) were laid on the triangulation map and oriented so that the orienting lines would pass through the triangulation points; the other points were then transferred by pricking, and the elevations indicated.

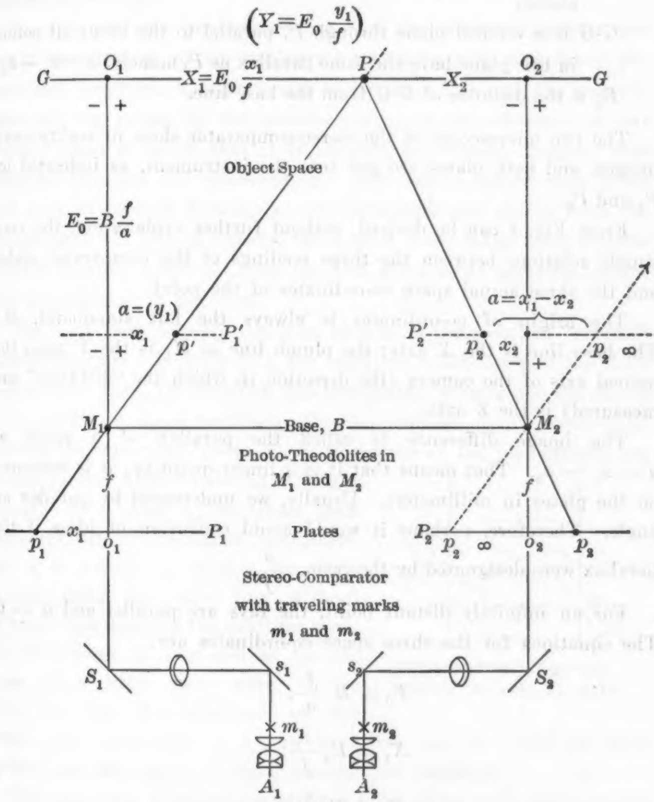
Having transferred several consecutive "point sheets," the drawing in of contour lines was commenced. Finally, the map was taken into the field for comparison with the actual topography, and objects which had been concealed from the camera, or had otherwise been missed, were filled in by free hand or by stadia, according to their size and importance. Also, on this last visit to the field, the topographic signs (buildings, culture, etc.) were indicated on the map.

Then the paper location was made on the original map or on tracings from it.

Theory of the Stereo-Photogram.

The theory of a stereo-photo-grammetrical exposure, in which both plates are in the same vertical plane, is illustrated by Fig. 1, in which M_1 and M_2 denote standpoints of the camera, that is, the center

of the objective. The difference of a few centimeters between center of objective and center of instrument can be neglected.



THEORY OF A
NORMAL STEREO-PHOTO-GRAMMETRICAL
EXPOSURE

FIG. 1.

M_1-M_2 ($= B$) is the horizontal projection of the base line;

M_1-O_1 and M_2-O_2 are the optical axes, and are parallel to one another and perpendicular to the plates and to the base line;

f is the focus of the objective;

P is any point to be determined;

p_1 and p_2 are the image points of P on P_1 and P_2, the negative plates;

G-G is a vertical plane through P, parallel to the base; all points in this plane have the same parallax as P, namely, $a = x_1 - x_2$;

E_0 is the distance of G-G from the base line.

The two microscopes of the stereo-comparator show in reality erect images, and both plates are put into the instrument, as indicated by P_1 and P_2.

From Fig. 1 can be derived, without further explanation, the very simple relations between the three readings of the comparator scales and the three actual space co-ordinates of the point.

The origin of co-ordinates is always the left standpoint, M_1. The base line is the X axis; the plumb line at M_1 is the Y axis; the optical axis of the camera (the direction in which the "distances" are measured) is the Z axis.

The image difference is called the parallax of a point, as $a = x_1 - x_2$. That means that it is a linear quantity; it is measured on the plates in millimeters. Usually, we understand by parallax an angle. Therefore, perhaps it would avoid confusion of ideas if the parallax were designated by the ratio, $\frac{a}{f}$.

For an infinitely distant point, the rays are parallel and $a = 0$. The equations for the three space co-ordinates are:

$$E_0 = B \frac{f}{a};$$

$$X_1 = E_0 \frac{x_1}{f};$$

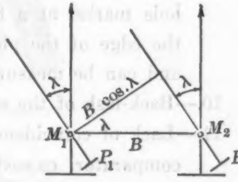
$$Y_1 = E_0 \frac{y_1}{f}.$$

It is essential in normal photo-grammetry that the two plates be in the same vertical plane. However, points can be determined from two plates not in the same plane, but, it might be mentioned, parenthetically, that in any case, with the instruments herein mentioned, a base line is used which is only from one-twentieth to one-tenth of the length of that required by the old photographic method.

The two plates are used in conjunction to determine the distance of a point, and the left-hand plate alone serves to determine its direction and elevation.

In principle, the photo-theodolite and the stereo-comparator are simply an extension of our ordinary binocular power of judging distance and direction; but the length of the base line and the magnification of the image by the microscopes raise the efficiency of our natural vision so much that it is possible to measure distance with sufficient accuracy for map-making. Although a single view gives only such an idea of distance as our experience can derive from the relative apparent size of objects, the second view, taken from a point removed laterally somewhat from the first, gives us an accurate insight into the structure of the view.

The most used, because the most simple, is the so-called "normal" stereo-photogram. Nevertheless, the scientific collaborator of the Zeiss Werkstätten, Jena, Dr. Pulfrich, has thoroughly investigated the theory of the curves of equal parallax. He has invented simple methods of plotting for exposures, where the optical axes of the photographs are equally turned either to the left or right (usually 30°), Fig. 2; therefore such exposures are of increased usefulness, and it is possible to double the angle covered from one base line without seriously complicating the drafting work. The field work is decreased at the expense of a little more work in the office, as the determination of the length and orientation of a new base line can be dispensed with. In this case, the curves of equal parallax are parabolas.



EXPOSURES WITH EQUALLY TURNED, OR PARALLEL, AXES
FIG. 2.

The exposures with horizontal axes inclined to each other at any angle, that is, convergent, if both axes are directed to one point in the foreground, is very similar to the old method of plane-table photogrammetry. This method, however, is seldom used, owing to the difficulties of plotting, which are not yet overcome. The curves of equal parallax are ellipses. The construction of one point requires too many auxiliary lines. It would go too far afield to enter here into a discussion of the accuracy of stereoscopic vision with our

two eyes, on the ground of recent physiological investigations. Also, the writer has not the space here to derive a complete theory as to the errors in measuring with the stereo-comparator; it may suffice to mention, among other various sources of error, the following:

1.—Inaccurate determination of the base line in the field;
2.—Inaccurate determination of the focus of the objective by the makers (all views are made with a fixed-focus camera);
3.—The lens not perfectly free from optical distortion;
4.—Too coarse-grained emulsion on the plates (which must be especially prepared);
5.—Not exactly vertical position of the plates during exposure;
6.—A slight inclination of the plates in relation to each other;
7.—A slight inclination of the axes in relation to the base line, though they are still parallel to one another;
8.—Not perfectly plane plates (especially with long-focus lenses);
9.—Expansion and contraction of the plates by change of temperature; or a creeping of the gelatine film, for a like reason, or by moisture. (A check against the latter is provided by two hole marks, at a fixed distance apart, which are printed on the edge of the plate automatically at the time of exposure, and can be measured later under the stereo-comparator.)
10.—Back-lash of the screws of the comparator;
11.—Lack of coincidence of the planes of the two pictures in the comparator, caused, for instance, by difference in thickness of the glass plates;
12.—One or both of the plates not resting exactly in a horizontal position in the comparator.

Obviously, it is necessary for the surveyor to understand and take into account how much each of these sources may affect the accuracy of the result. It is necessary and possible to be able by checks to determine the sources of errors and make proper adjustments accordingly.

INSTRUMENTS.

The stereoscopic method is very sensitive, and the instruments used must be of very great precision. The only manufactory of these instruments is the well-known Zeiss Werkstätten, Jena. The instruments

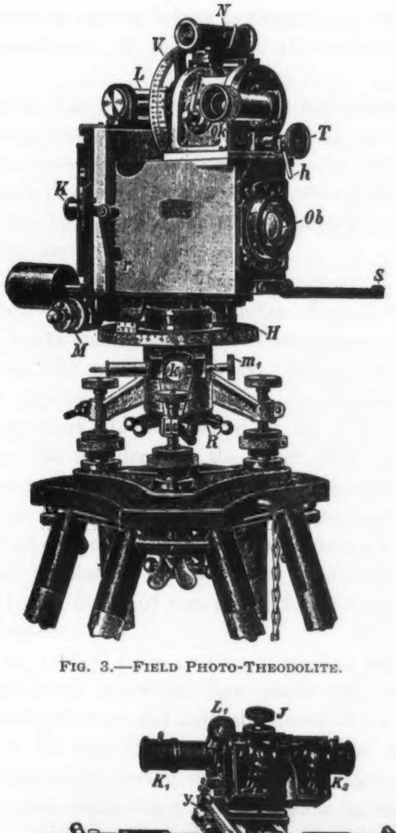
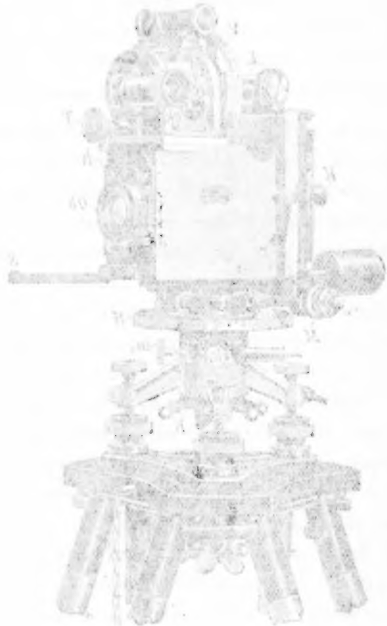
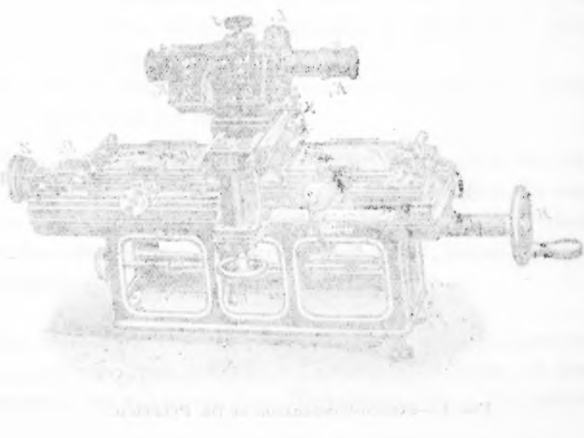


FIG. 3.—FIELD PHOTO-THEODOLITE.

FIG. 4.—STEREO-COMPARATOR BY DR. PULFRICH.





used in this particular survey belong to the geodetic outfit of the Civil Engineering Department of the Chinese Government University at Peking.

The equipment, Fig. 20, consisted of one field photo-theodolite, for 9 by 12-cm. plates, focus 98.54 mm.; three tripods; one measuring rod for optical base measurement in connection with the directing telescope of the photo-theodolite; twenty double plate-holders; one stereoscopic range-finder (power $4x$); one stereo-comparator, Model D; and a special drawing device.

The Field Photo-Theodolite.—This instrument is a combination of a metallic box camera with a theodolite (Figs. 3 and 5), so that the particular conditions of a "normal" stereogram can be fulfilled expeditiously and with extreme accuracy. An error of from 1′ to 2′ in the relative position of the plates to one another is much too great. Then the marks to be adopted in the back frame in front of the plate must be considered as the image of an infinitely distant point, from which the parallaxes are to be measured, for instance, exactly to 0.01 mm., if known fixed points in the landscape are to be dispensed with and corrections avoided. The highest admissible angular deviations in the position of the two plates to one another, therefore, are determined by dividing 0.01 mm. by the focal length, $f = 180$ mm., or 10″, approximately.

Instead of the vertical hairs previously used in the camera frame for photo-grammetrical exposures, new marks are produced which, in an unmistakable manner and with the necessary accuracy, furnish a fixed point for the zero of the values of parallax and for finding the optical and horizontal axes of the plate when taking measurements from it on the stereo-comparator. It is possible to make changes in the adjustment of the camera to the horizontal and to the base line, without the assistance of the marks in the camera frame, at any time after having inserted the dark slide. The adjustment of the two plates in one plane—to be done with an accuracy of about 10″—is not only performed with ease and precision, but is accessible for immediate examination and rectification, right up to the very moment of exposure.

The photo-theodolite proper consists of three principal parts which are in adjustable position to one another: the central bearings, with the horizontal circle, insertable in the tripod, the camera firmly fixed

to the axis of rotation, and the pointing telescope with the vertical circle.

The horizontal and vertical circles are divided into half degrees, the vernier permitting readings of 1′ and estimations of ½′. The horizontal circle is adjustable—coarse by hand and fine by micrometer screw—for the purpose of setting on a certain zero point.

The vertical axis of rotation can be firmly clamped and fine adjustment effected with a micrometer screw, which is also designed for the measurement of the base.

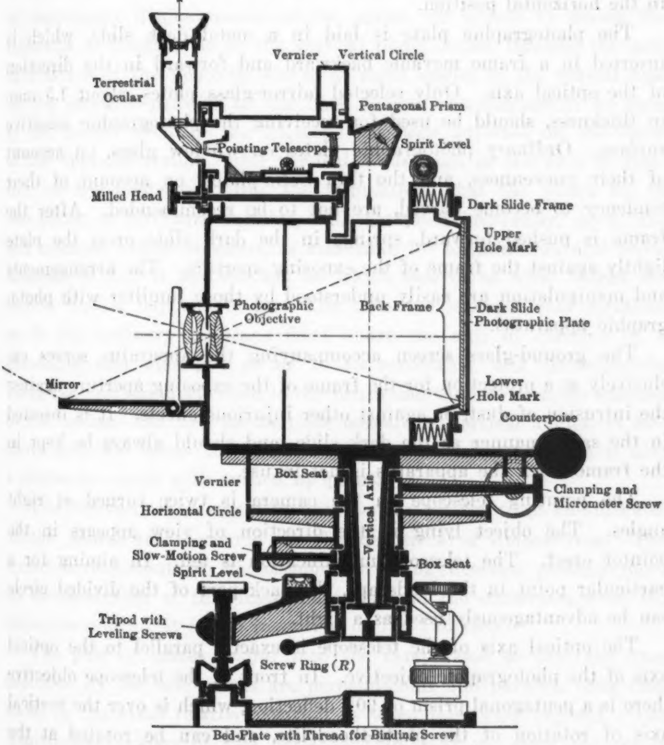
The camera casing is of light metal, cast in one piece, into which the objective is fixed in front; the back frame for the sensitive plate is behind. The optical axis is at right angles to the plane of the back frame at a certain point mentioned later.

The ortho-protar objective in the photo-theodolite is particularly suitable for photo-grammetrical and stereo-photo-grammetrical exposures, on account of its simple structure and consequently its ability to resist external physical influences, its fine definition from the center to the margin of the plate, and its well-nigh perfect orthoscopy—the deviations are demonstrably smaller than 0.01 to 0.02 mm. The horizontal image angle is more than 60°, so that in wide-angle views only six exposures—in which the margins somewhat overlap one another—are necessary for the whole circuit. The immovable objective is below the middle of the exposing aperture of the back frame. In cases where only one photo-theodolite is used, the objective is provided with a fixed stop of $\frac{f}{40}$. The focal length of the objective is most carefully determined in the works, up to 0.1 mm. for each instrument, and stated on delivery of the apparatus.

The back frame is ground perfectly plane, and adjusted so that all around the photographic plate a few millimeters are blocked out. It is exactly parallel to the axis of rotation of the photo-theodolite.

The new marks in the back frame are to be seen on the photographs, Figs. 18 and 19. They consist of two circular holes, with conical embrasures, turned toward the objective, about 0.15 mm. in diameter, one of which is in the middle of the upper margin of the frame, the other in the middle of the lower margin. The line joining the two holes is directed parallel to the axis of rotation of the photo-theodolite, and cuts the optical axis in one point.

The hole marks have this advantage, over the knife-edges generally used, that their centers undergo no displacement whatever through differences in illumination, and can be easily rediscovered under the microscope of the stereo-comparator with the necessary accuracy.



DIAGRAMMATIC VERTICAL SECTION THROUGH THE PHOTO-THEODOLITE
(Pointing telescope is drawn with the ocular in the vertical position)

FIG. 5.

Because of the adjustment mentioned, the vertical axis of rotation of the photo-theodolite, the optical axis of the camera, and the horizontal of the plate—the last is determined by the reading on the *y*-scale and by the slide rests of the stereo-comparator lying at right

angles to one another—form a fixed rectangular co-ordinate system. There is required then only the exact vertical setting of the axis of rotation, which is done with the set-screws on the tripod and the 10-in. spirit level on the camera, and then the plate lies in the vertical plane, and the optical axis and the horizontal axis of the plate are in the horizontal position.

The photographic plate is laid in a metal dark slide, which is inserted in a frame movable backward and forward in the direction of the optical axis. Only selected mirror-glass plates, about 1.5 mm. in thickness, should be used for receiving the photographic sensitive surface. Ordinary photographic plates on window glass, on account of their unevenness, and the thin Solio-plates, on account of their tendency to become curved, are not to be recommended. After the frame is pushed forward, springs in the dark slide press the plate lightly against the frame of the exposing aperture. The arrangements and manipulation are easily understood by those familiar with photographic apparatus.

The ground-glass screen accompanying the apparatus serves exclusively as a protection for the frame of the exposing aperture against the intrusion of dust, or against other injurious effects. It is inserted in the same manner as the dark slide, and should always be kept in the frame when the apparatus is not in use.

The pointing telescope on the camera is twice turned at right angles. The object lying in the direction of view appears in the pointer erect. The telescope magnification is ten. In aiming for a particular point in the landscape, the back part of the divided circle can be advantageously used as a sight.

The optical axis of the telescope is exactly parallel to the optical axis of the photographic objective. In front of the telescope objective there is a pentagonal prism of 90° deflection, which is over the vertical axis of rotation of the photo-theodolite, and can be rotated at the same time as the twice-turned terrestrial ocular about the optical axis of the telescope. The telescope objective and the cross-lines take no part in the rotation. These stand in connection with the pentagonal prism and the ocular only through the roof of the camera chamber, and form a perfectly closed-in pointer telescope, secured against lateral displacements by screws and pins and against external influences by the hollow body carrying the prism and ocular. With

the assistance of this pointer, the direction of the optical axis of the photographic objective can at any moment be fixed.

In consequence of these arrangements, the pointer is simultaneously available as a sighting theodolite telescope, and, by virtue of the constant 90° deflection to the optical axis by the prism, independent of contingent displacements of the prism, all points to which the telescope is directed lie in the vertical plane drawn through the axis of rotation of the photo-theodolite at right angles to the optical axis of the camera—parallel to the back frame. Consequently, as soon as the double circle of the telescope has been adjusted on the visier of the opposite station, after vertical adjustment of the axis of rotation of the photo-theodolite, the stipulation, that the plates lie in one plane, is forthwith fulfilled, and, as stated previously, up to the last moment before exposure, is accessible for examination and rectification, so that it matters not on which side or at what height the second station happens to be. The telescope is capable of rotation, as in the ordinary telescope fixed above the axis of rotation of a theodolite and arranged so as to be revolvable, and therefore—in using the vertical circle fixed to the prism and the two verniers adapted to the casing—can also be used for measuring vertical angles, to 30° above and 30° below the horizon. Above the telescope there is also a magnetic needle with turned up point.

The Stereo-Comparator.—By this instrument the three co-ordinates of the points are measured on the negatives or diapositives. The recent development of stereoscopic surveying depends perhaps more on the invention of this instrument than on the construction of the photo-theodolite. It is the invention of Dr. Pulfrich.

The pair of plates is put into the comparator so that the axes indicated by the hole marks are perpendicular and parallel, thus they stand in the same relative position as when they were exposed. The comparator consists of two parts; a framework to carry the plates and a binocular microscope for viewing them.

In detail, the construction of this instrument, Fig. 4, is as follows: A heavy bed, *A*, has a track on which the plate-carrying frame slides in a lateral direction. Surmounting this is a second track, *B*, at right angles to the first, on which the microscope carriage slides. These motions serve for measuring the two co-ordinates which are parallel to the plates. The remaining co-ordinate (E_0, Fig. 1) is

determined by measuring the parallax by an independent lateral motion of the right-hand plate.

In placing the plates, P_1 and P_2, in the frame, they are adjusted so that the lines connecting the hole marks at the top and bottom of the plate are parallel to the track, *B*. For this purpose, the plate-holders are mounted on the plate carriage so that they can be turned through a small angle by slow-motion screws, D_1 and D_2. In order to compensate for the difference in elevation of the two standpoints from which the plates were taken, the right-hand plate is adjustable vertically by the screw, *C*.

Lateral deviation is measured by the scale, *X*, vertical deviation by the scale, *y*, and distance, that is, parallax, by the scale, *Z*, to 0.001 mm. The parallax measurement must be made with the utmost accuracy, as the other two co-ordinates are derived from it. Its importance is analogous to that of quick and accurate distance measurement in tacheometry.

For viewing the plates there is a Helmholz tele-stereoscope in connection with a binocular microscope (power $6x$ and specific plastic $4x$). In order to be able to measure with the tele-stereoscope in the image plane of both eyepieces in the center of the field, a balloon-shaped measuring mark is fixed. This mark resembles a solid circle standing above a plus sign. When properly adjusted, the stereoscopic effect causes these two images to coincide and appear as a single image floating in space. With the shifting mechanism, the observer is able to place this mark on any desired point in the landscape. By turning *H*, it appears to move laterally; by turning *V*, it appears to move vertically; by turning *Z*, it appears to move to and fro. This artificial mark has the function of the rodman in a tacheometrical survey, with this difference, that it depends solely on the will of the observer and not on the capacity of the rodman; also, it is not influenced by wind or weather, and is not handicapped by roughness and inaccessibility of the territory to be surveyed.

It is important to notice that it is not necessary for the observer to have the power of stereoscopic vision, because, in placing the balloon mark on a certain point in the landscape, he first sets the point in the left ocular, then brings the mark into coincidence in the right ocular, before making a binocular observation, as in the case of those with one defective eye or with one eye especially trained.

Plotting Device.—The device for the plotting of the map (Fig. 6) is a strong mirror-glass plate on which a steel ruler, L_1, is pivoted at a point representing the left standpoint, M_1. There is also a ruler, L_2, which can be clamped in a direction parallel to the optical axis. Both are of thin saw-blade steel.

The Range-Finder.—The stereo-comparator has an advantage over another very useful surveying instrument, the Zeiss range-finder. The latter, invented and perfected by Dr. Pulfrich, was found very useful in checking the side measurements of the triangulation, the check measurements on the more important points in the photographic survey, and also in the preliminary measurement of distances in order

DRAWING-GLASS-PLATE FOR PLOTTING STEREOGRAMS

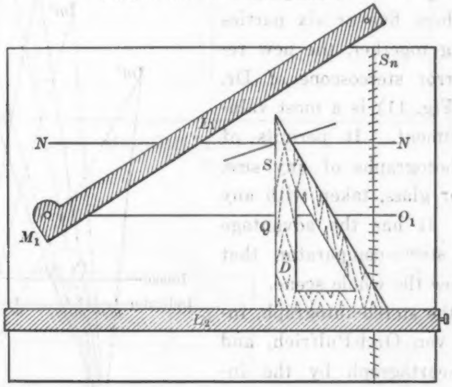


FIG. 6.

to make a proper selection of the base lines for the photographs, etc., taking into consideration the scale of plotting. The observer sees in the range-finder a stadia scale assuming the appearance of a solid zigzag line floating in space (Fig. 7). By this scale he is enabled to determine the distance of any point, "as if counting the poles of a telegraph line." He brings into coincidence with the point in question that mark on the scale which is apparently at the same distance. The range-finder in its usual form has a fixed scale.

The Stereo-Micrometer.—Another quite useful instrument, but of more theoretic-pedagogic interest, is the stereo-micrometer. It serves exceedingly well to explain the principle of the "wandering

or traveling mark" (Figs. 9 and 10) and the stereoscopic method of measuring distances in general. In this micrometer there is a "wandering mark" and pictures in fixed position relative to each other; in the stereo-comparator there are two fixed marks under which the plates are movable. The effect is practically the same.

Accuracy in working with the range-finder depends very much on the capability and practice of the observer. It might be mentioned that the ordinary stereoscope can only be used for small pictures taken with an ordinary camera with objectives separated by the usual interpupilary distance of 65 mm. Also, it might be mentioned here that, for greater stereographic surveys, where five or six parties are working together, the new reflecting-mirror stereoscope of Dr. Pulfrich (Fig. 11) is a most valuable instrument. It permits of viewing photographs of any size, on paper or glass, taken with any base line. It has the advantage over the stereo-comparator that one can view the whole scene.

FIXED MARKS IN THE ZEISS RANGE-FINDER

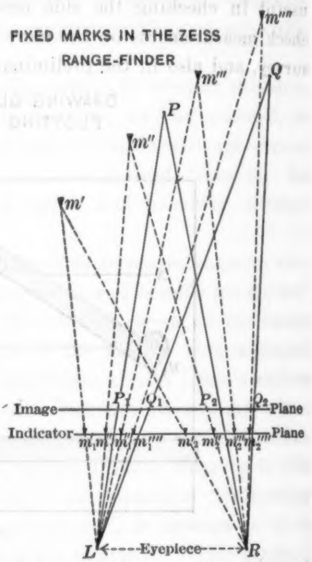


Fig. 7.

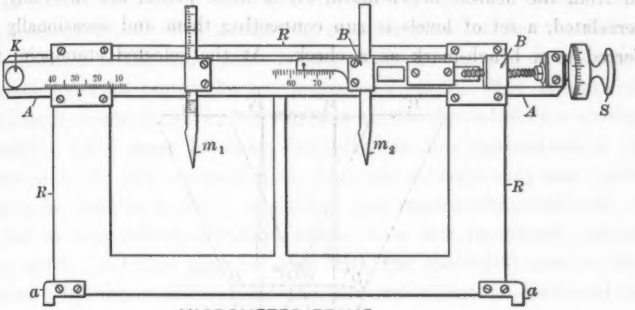
About the stereo-autograph, invented by von Örel-Pulfrich, and the stereo-cartograph by the inventor of the comparator, something may be said later, because they do not belong to the ordinary surveying outfit on account of their high cost.

General Methods of Work.

It is necessary to distinguish between the work done in the field and that done in the office.

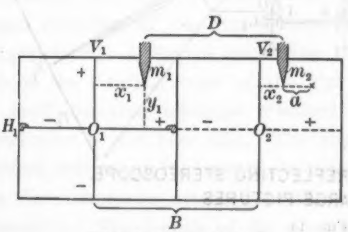
Field Work.—The outdoor work consists in the preparatory survey, that is, in selecting suitable and elevated auxiliary points. It is frequently desirable to run a line of bench-mark levels through the district to be surveyed. These can be referred to when necessary.

The natural characteristic points in the landscape, or the artificial signals to be erected, must be situated so that the resulting intersections are favorable for the triangulation; on the other hand, it is necessary that three, or better more, are visible from each standpoint, for the purpose of orientation and eventually for taking sights to the elevation marks.



MICROMETER FRAME

FIG. 8.



PRINCIPLE OF THE TRAVELING MARK

FIG. 9.

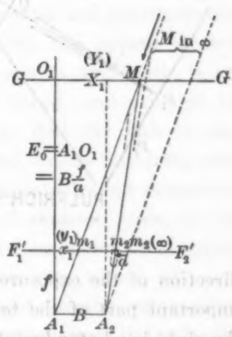
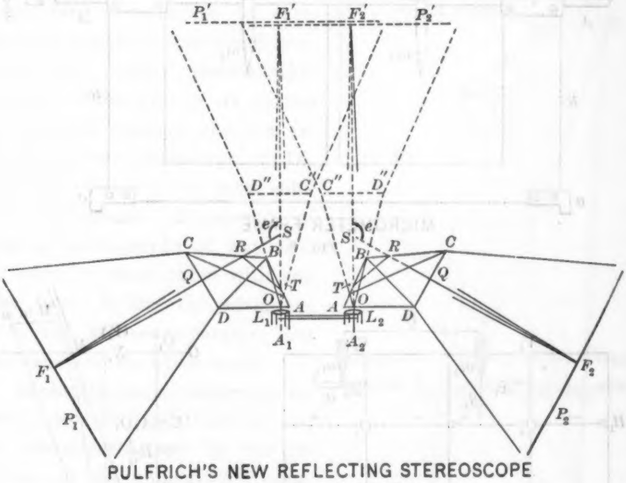


FIG. 10.

It is very desirable, but not always possible, to have such elevation control points on each plate. In selecting the trigonometrical points, the plan of attack with the camera must be considered beforehand. Occasionally, it is better, or even necessary, first to select the photographic standpoints, in order to be sure that they can be oriented in the auxiliary net thrown over the territory by resectioning or other-

wise. It is desirable, of course, to have these points visible on the plate. The erection of characteristic signals, as in first-class triangulation, would be too expensive for smaller distances. It is better, therefore, to erect simple poles with flags on the higher hills, so that they are clearly silhouetted against the sky and become easily visible on the plate. The selected photographic standpoints are leveled in from the nearest bench-mark, or, if these points are favorably interrelated, a set of levels is run connecting them and occasionally referred to a bench-mark as a check. At the selected standpoint the



PULFRICH'S NEW REFLECTING STEREOSCOPE FOR LARGE PICTURES

Fig. 11.

direction of the exposure is determined, and the distance to the most important part of the territory is taken with the range-finder. Then the right-hand standpoint is fixed at the length of the base in a direction perpendicular to the optical axis of the first exposure.

The base line bears a certain ratio to the average distance of the chief objects in the view and to the scale on which the map is to be plotted. Nevertheless, one need not be too strict, as the natural lay of the land usually renders a little more or less distance necessary. It is desirable that all stereograms of any one survey be of uniform accuracy.

It has proved very useful to mark these standpoints carefully. This makes it possible to return to the field and repeat exposures which have not proved entirely successful photographically, or to choose more favorable conditions of weather and light, without the necessity of repeating auxiliary measurements.

The photo-theodolite on the left standpoint gives the direction to all visible signals; on the right standpoint, higher or lower, is set up a second tripod with a sighting cone in place of the theodolite. The direction to the right point is best chosen at the zero or reference direction, and all the angles are measured from it. The azimuth of the base is taken either with a compass on the theodolite or a special compass. This must be done carefully, as this determination of direction is far less accurate than the angle measurement and ought only to be used as a check or guide. The opportunity ought not to be lost of also taking elevation angles to a few prominent signals as a check. All this may be done with the theodolite part of the camera. By these observations the left standpoint is connected to the control, in position, elevation, and orientation. At this time the base line measurement can be made. Direct steel tape measurement is possible with a short length of favorable ground and approximately equal elevation of the standpoints, but usually the optical method is preferable. A special measuring lath, 1 or 3 m. in length, is set up on the sighting cone of the right-hand tripod, and oriented by a small directing telescope attached to it so that the lath is perpendicular to the base line. The angle under which this lath, or a certain portion of it, appears, is measured repeatedly and accurately by the micrometer attached to the horizontal tangent screw of the theodolite. The length of the base line must be determined within 0.1%, corresponding to the parallelism of the axes within 10″, and to the accuracy of the parallax measurement in the comparator. According to the previously mentioned equations, an error in the length of the base enters proportionally and directly into all co-ordinates. The elevation angles to the right standpoint having been taken, the reduction of the base line to the horizontal can be done in the office. It must be expressly mentioned that the tripods should be set firmly. In hard and rocky soil, or in elevated positions exposed to the wind, extreme caution should be taken in this matter. Finally, the height of instrument (averaging 1.30 m.) from peg to center of objective, should

be measured. Having completed these auxiliary measurements, the actual exposure of the plate can be made, if the weather and sun permit.

The numerating device in the camera is set so as to record photographically the proper number of the exposure. Then the plate-holder is inserted, the slide is removed, and, by a special device, the plate is brought to an even contact with the plain brass rim of the back of the camera chamber, and locked in this position. Just before unstopping the lens, a final observation is made with the telescope pointing to the sighting cone at the other end of the base line, to make sure that the plate is in its proper plane. Immediately thereafter, the exposure is made, with all precaution against disturbing the apparatus.

It is advisable to make all the observations carefully, and always in the same sequence. An omission may necessitate remeasuring. After the exposure, the plate is drawn from its seat on the brass frame, the slide is inserted, and the holder is removed from the camera. Additional exposures can then be made at 30° with the first, or, if these are not desired, the camera and sighting cone are interchanged between the two ends of the base line. This is done without disturbing the tripods. On the right-hand standpoint the exposure can be be made without making further measurements, as the camera has been leveled and oriented.

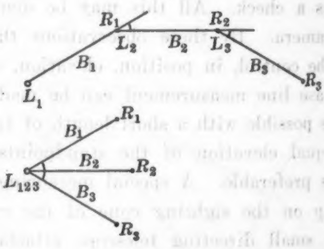


FIG. 12.

In order to make use of some especially favorable elevation for taking a panoramic survey, it is possible to use a series of base lines, either connected in a simple polygon or radiating from a single point (Fig. 12). In the latter case, the left standpoint serves for all base lines, and, of course, very greatly simplifies the field work; this, however, is not always possible on account of the lay of the land, as the selection of even a single base line is sometimes difficult.

A third possibility, recently used quite frequently, is, as already mentioned, making photographs equally inclined (usually at 30°)

to the base line, but still with the optical axes parallel. The writer always works with the horizontal axis, and prefers rather to shift the objective vertically, in order to make full use of the plate, or to bring the important features to the center of the plate, where the drawing is most accurate. It is best to set off the 30° deflection angle from the zero point of the circle, so that any error in the division has no influence on the parallelism of the axes, which must also be assured within 10″. It may be mentioned that, according to the investigation of von Hübel and Pulfrich on stereograms with parallel axes, a small deviation of both axes in the same direction gives a much smaller error than a convergence. Rough sketches, to indicate the territory included by an exposure, are convenient guides in making subsequent exposures during the interval before the development of the first ones. This will prevent the omission of narrow strips, or too much overlapping of consecutive pictures. With this the chief part of the field work is finished.

In surveying railway lines in mountainous districts, one may distinguish two principal cases:

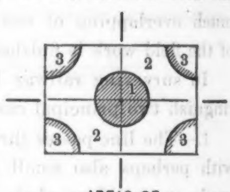
1.—The line passes through a fairly straight and broad main valley, with perhaps also small side valleys, and is on a fairly continuous grade on one slope of the valley. This is the most favorable case for the photographic survey, as the work can be done from the opposite side of the valley, with the different pairs of photographs, taken in approximately the same direction from nearly the same level.

2.—The line passes through a very tortuous valley, or across the topography, so that it frequently crosses the valleys. In this case the photographs must be made in all directions, and frequently from stations where it is difficult or impossible to include all points, especially those in the bottom of the valley, without an expenditure of too many plates which cover only small areas. In such a case it is best to commence the field work with a tacheometric survey of the lower ground, and, with this as a neucleus, to spread the survey by photographic means over the remaining higher territory. A combination of a rapid continuous tacheometric survey of the river with a photographic survey of all the higher and steeper parts is the most economical method. The contour lines are easily and safely developed from the natural guide line—the course of the river—thereby giving ample opportunity to check on the controls. There will be standpoints

near the bends where photographs can be taken looking into and over the valley for certain distances.

Office Work.—First, the triangulation must be calculated and plotted, and the trigonometric levels determined. The net having been plotted, say, on a scale of 1: 10 000, copies of it should be issued to each field party, when several are simultaneously at work, in order to avoid confusion. The base lines, elevations of standpoints, orientation angles, etc., are calculated in the office. The standpoints must be denoted clearly and specifically in order to correspond to the numbers registered on the plates by the camera, independently of the plate-holder numbers. Base line No. 7, for instance, has a plate pair with the numbers I-3 and I-4, so that 50 base lines and 100 plates can be denoted by the numerating device.

In the purely photographic part, such as developing, intensifying, or reducing the negatives, making diapositives, prints, enlargements, etc., the photographer has to aim at clearness and accuracy, rather than artistic effect. If the staff is not skilled in photography, it is best to do this work in the field, in order to be able to replace spoiled plates without delay.



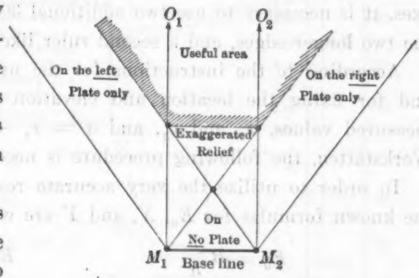
AREAS OF DIFFERENT ACCURACY
FIG. 13.

The chief work in the office is the measuring of the plates under the stereo-comparator. After a short practice it is possible to measure directly from the negatives. They are more exact, as all reproductions must lose in accuracy and sharpness. Nevertheless, it is advisable to make diapositives, first, in order to insure against the loss or breakage of the negative, and secondly, because sometimes it is possible to obtain good diapositives from weak negatives.

The value of a plate depends on the following considerations: All points which appear on one plate only, though that plate may be a good one, cannot be measured, as they cannot be viewed stereoscopically. Figs. 13 and 14 show the areas of relative accuracy and usefulness on any plate. The points of the very near foreground do not appear on any plate. For instance, those on the right plate and the left border of the left plate are useless. Those of the farther foreground show exaggerated relief, and are not suitable for measuring, as the parallax is too great. Points at very great distances become too in-

accurate, but can eventually be used for the mere topographical completion of the map.

In measuring the plate, the control points are considered first, if they are visible on the plate; and their co-ordinates are measured and compared with those on the triangulation map. If they are not visible, the distance, lateral deviation, and elevation are taken from the map, and the comparator is set to the corresponding scale readings, in order to see where the point falls in the view. Occasionally, it may happen that the image of the point can then be discovered near or coinciding with the balloon mark. At least, it can be observed whether the balloon mark assumes a position on the surface of the ground, or an impossible position in the air, or beneath the surface. Disagreements or errors must be first explained or removed; such precautions are absolutely necessary in order to avoid absurdities. After making sure that the check points agree, the measurements for the other points can be taken.



USEFUL AREA OF A NORMAL STEREO-EXPOSURE

FIG. 14.

In selecting the points needed for drawing the contours, either of two principles may be adopted: All points of equal parallax may be taken, or points of equal y-reading. This simplifies the work of noting and plotting by the elimination of one variable, in each case. The chief objection to such a method is that it is not flexible enough. It is similar to the old cross-profile method. The other principle is to make, in the shelter of the office, a tacheometric survey. This requires the judgment of the experienced topographer, and is a little more troublesome, as all three variables must be considered. However, this method was preferred by the writer for "normal" stereograms. When working in shifts, or in any case to avoid repetition or confusion, the points taken out may be marked on a print in red. The observer with the comparator must consider the scale of the map, and must in imagination travel into the scenery and select fairly evenly dis-

tributed points. Though the parallax can be read to 0.001 mm. and the x- and y-scales to 0.01 mm., it is useless to read to a greater degree of accuracy than can be plotted on the map.

A numbered list of the points, with their co-ordinates, is made and from it the points are plotted.

The plotting device consists of a glass plate, about 1 m. square, with paper pasted on both sides (Fig. 6). On this are drawn the divisions required for the construction of the points (Fig. 15). There is a metallic ruler, L_1, with millimeter divisions on the upper edge and pivoted at M_1 on a conical bearing attached to a metallic inlay in the glass plate. There is also a ruler, L_2, which can be clamped to the plate, and a triangle, D. It is arranged so that the plate can be used on either side. For the plotting of stereograms with inclined axes, it is necessary to use two additional 30° triangles with scales on the two longer edges, and a second ruler like L_2.

According to the instructions for the use of this drawing device, and for fixing the location and elevation of a point, P, from the measured values, x_1 and y_1, and $a = x_1 - x_2$, issued by the Zeiss Werkstätten, the following procedure is necessary:

In order to utilize the very accurate readings of the comparator, the known formulas for E_0, X, and Y are written as follows:

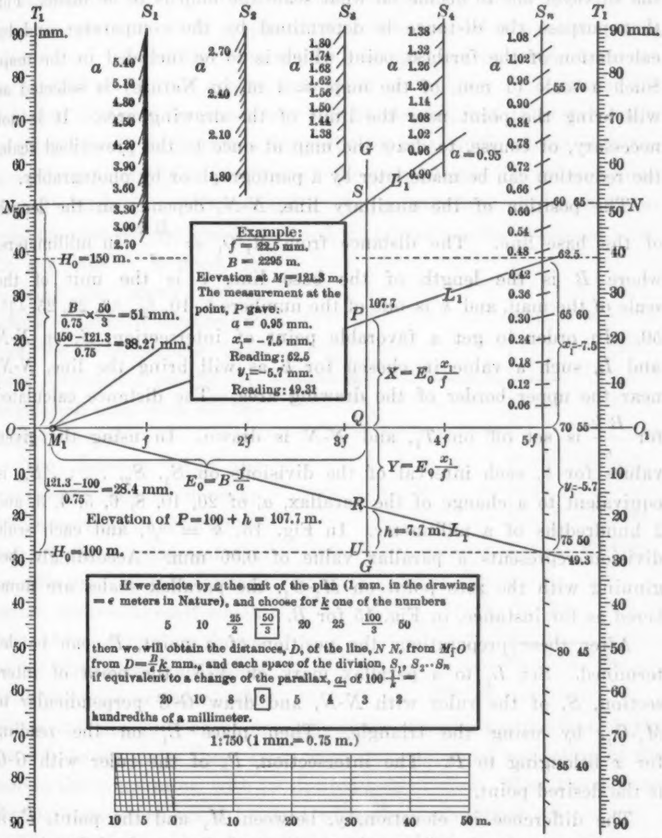
$$E_0 = B\,\frac{f}{a} \qquad\qquad E_0 = k\,B\,\frac{n\,f}{k\,n\,a}$$

$$X = E_0\,\frac{x_1}{f} \qquad\qquad X = E_0\,\frac{n\,x_1}{n\,f}$$

$$Y = E_0\,\frac{y_1}{f} \qquad\qquad Y = E_0\,\frac{n\,y_1}{n\,f}$$

The coefficients, k and n, will be explained later. M_1 always represents the left-hand station, M_1-O_1 the direction of the optical axis. Lay off from M_1 on M_1-O_1 (Fig. 15) the distance, f, the focus, in millimeters, repeated to the edge of the drawing. Through the points thus obtained draw perpendicular lines, S_1 S_2, . . . etc. Divide these S-lines so that the divisions on $S_1 = 1$ mm., on $S_2 = 2$ mm., on $S_n = n$ mm., etc. The last line is divided throughout its length, the others only at the upper extremity. T_1 and T_1 are two millimeter scales with zero points on M_1-O_1. Thus each plate is prepared once for all. The actual plotting is done on tracing paper mounted over this. The cone pivot of the ruler, L_1, remains un-

covered. The scale, S_n, is now numbered for x and also for y, but the point on M_1-O_1 is not denoted by zero. The balloon mark of the comparator is set to cover the hole marks on the left topographic



LOCATION AND ELEVATION OF A POINT, P, BY MEANS OF THE MEASURED VALUES, x_1, y_1, $a = x_1 - x_2$

FIG. 15.

plate and then the verniers of the scales, x and y, are adjusted to read whole numbers. These readings are then the respective x- and y-values to be given the middle point of the scale S_n (70 and 55 in

Fig. 15). Starting up and down from this point, the remaining points of S_n are numbered. Both rows of figures are written on the right side of S_n. The figures on the left are to denote the parallax. Now the surveyor has to decide on what scale the map is to be made. For that purpose the distance is determined by the comparator and by calculation of the farthest point which is to be included in the map. Such a scale (1 mm. of the map = 1 m. in Nature) is selected as will bring the point near the limit of the drawing area. It is not necessary, of course, to draw the map at once to the prescribed scale, the reduction can be made later by a pantograph or by photography.

The position of the auxiliary line, N-N, depends on the length of the base line. The distance from M_1-$O_1 = \frac{B\,k}{c}$, in millimeters, where B is the length of the base line, c is the unit of the scale of the map, and k is one of the numbers 5, 10, $\frac{25}{2}$, $\frac{50}{3}$, 20, 25, $\frac{100}{3}$, 50. In order to get a favorable point of intersection, S, on N-N and L, such a value in chosen for k as will bring the line, N-N, near the upper border of the drawing area. The distance calculated for $\frac{B\,k}{c}$ is set off on T_1, and N-N is drawn. In using the given values for k, each interval of the divisions on S_1, S_2, ... S_n, is equivalent to a change of the parallax, a, of 20, 10, 8, 6, 5, 4, 3, and 2 hundredths of a millimeter. In Fig. 15, $k = \frac{50}{3}$, and each scale division represents a parallax value of 0.06 mm. Accordingly, beginning with the zero point on M_1-O_1, the parallax scales are numbered as for instance, in Fig. 15 for B.

After these preparations the position of a point, P, can be determined. Set L_1 to a parallax mark on N-N, the point of intersection, S, of the ruler with N-N, and draw G-G perpendicular to M_1-O_1, by using the triangle. Then place L_1 on the reading for x belonging to P. The intersection, P, of the ruler with G-G is the desired point.

The difference in elevation, y, between M_1 and the point, P, is then determined. Set L_1 at the reading of the y-scale belonging to P, mark R of the ruler, L_1, with G-G, and measure R-$Q = y$, by compass or glass scale. The absolute elevation, H, of the point, P, can be determined as done in the figure by a few auxiliary lines parallel to M_1-O_1 directly on the drawing. Another means of determining H directly is the millimeter division on the edge of the

steel triangle. The elevations are noted on a slip of paper beside the division, and the ruler, L_2, is placed so that the divisions in all positions of the triangle passing along L_2 are cut by the axis, M_1-O_1.

Sometimes the ordinates are better calculated if the progress of the work in plotting with graphical determination of the elevation is slower than that of the comparator readings, and an extra man can be engaged on it. Of course, one could try at once to construct the contours on these tracing paper "point sheets", but it is preferable to prick off all these points from the various single sheets into the general map. Certain portions of the ground may be covered from several standpoints. Occasionally, the points from different sheets may supplement and check each other; others can be dispensed with. In any case it is good, in working out many stereograms of difficult country, to plot the photographic base lines beforehand on the triangulation map with all the angles covered.

The observer using the comparator consults the map before starting work on a new pair of plates, takes out the co-ordinates of the control points, assures himself which parts of his section are shown on his plate alone and which by others once or twice. Of course, it is not quite easy, especially if the observer has not been in the country, to recognize these areas from the orthogonal projection of the map in the comparator, with its limited field of view. Here the new reflecting stereoscope could be used with great advantage.

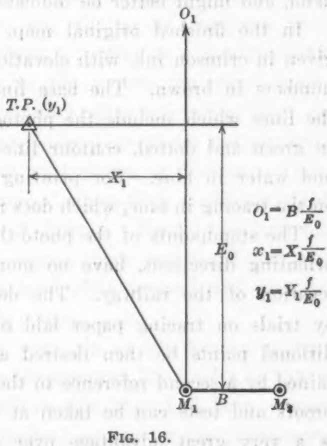


FIG. 16.

Characteristic points in a doubly or triply covered section are extremely well suited for special control checks of base lines.

As mentioned previously, it is not absolutely necessary to observe the two plates simultaneously, as the balloon mark of the left, and afterward that of the right, microscope can be successively set on a point; but it is highly desirable that the observer be able to see

stereoscopically, as he is thereby enabled to place the mark as conveniently as though he were able actually to wander at will through the country represented in the photographs.

In photographic surveying no special methods of drawing the contours are involved, as they are filled in from the points obtained, just as from those obtained by any method of surveying. In a preliminary railway survey one usually has to deal with a strip of country from 300 to 700 m. in width, but, in general topographic and geological surveys one has to measure extended areas. For the territory of the most probable line, the greatest accuracy, relatively, is necessary, but, for the more distant country, only a rough approximation is needed. Such supplementary work can be done in the field by freehand, and might better be indicated by dotted lines.

In the finished original map, the auxiliary triangulation net is given in crimson ink, with elevations indicated in red, and the control numbers in brown. The base lines are drawn in solid green lines; the lines which include the photographic angle of each exposure are in green and dotted, contour lines are in brown, buildings in black, and water in blue. For printing the auxiliary lines may be drawn on the tracing in blue, which does not print out.

The standpoints of the photo-theodolite, the base lines, angles, and orienting directions, have no more direct interest after the definite location of the railway. The definite line is determined as usual by trials on tracing paper laid over the original map. Should additional points be then desired at certain places, they may be obtained by a second reference to the original photographic plates. Also proofs and tests can be taken at any time when doubts arise. This is a very great advantage over the ordinary tacheometric method, where a revision of the map means an entire remeasurement in the field. It is even possible to seek the single points of the located line in the office by using the plates, as the map gives the parallax and the lateral deviation. For a certain value of *a* or *x*, it is only necessary to set the balloon marks on the ground in order to obtain directly the ordinate required for the plotting of the profile. In this way all inaccuracies, produced by the more or less free ideal design of the contours, are avoided.

The possibility of plotting on a larger scale certain places, where perhaps structures are to be erected, depends on whether the length

of the base line is sufficient. Cross-sections, also, can be easily taken. Finally, it is possible to transfer points of the line backward from the map into the photographic plates or prints, so that, in the pictures one may follow the proposed line over hill and valley.

The Stereo-Photo-Grammetric Survey of the Valley Near Ichang.

The foregoing explanations are given in great detail on account of the newness and possible importance of the subject. They deal with the general working plan, the theory of stereo-exposures, the instruments used, and the methods. Something may now be added about the survey of the rocky valley near Ichang.

Not far from Ichang commence the famous gorges of the Yangtze Kiang, noted among travelers, but feared by navigators. The locality and conditions for the survey were very similar to those met in the building of the Bagdad Railway, where, according to a communication from Ph. Holzman and Company to the writer, the stereo method was also used. The Ichang is a winding, deep-cut valley, with canyons and gorges. The scenery is extremely picturesque and attractive, especially to one arriving from the vast plains of Northern China. The slopes are clothed with semi-tropical vegetation, sometimes thinly, and sometimes with dense masses of foliage. Needle oak and high ferns abound. In the vicinity of houses there are single palms, in the bottom of the valley occasional level plots covered with rice paddies, and on the flatter slopes a little cotton.

The collaborators of the writer were six of his former students, just graduated from his class at the Government University at Peking, and speaking English. Shortly before this they had been engaged with the writer on another (but smaller) survey by the photo-stereographic method. These men acted as party heads. The three German engineers were not acquainted with the method. The ten students from Hupeh did not speak a European language, and were not acquainted with German instruments in general, or with the stereographic method.

Unfortunately, the writer could make the survey only during his vacation, namely, in July and August. These months are in every respect the most unfavorable ones in the Yangtze Valley, which, even at the best, has a bad reputation climatically. The heat is perhaps not considerably higher than it may be occasionally in North China,

but the greater humidity causes greater discomfort. Even with a clouded sky, the atmosphere is extremely sultry. In this glowing valley it is practically impossible for a European to work in the direct sun between the hours of 11 A. M. and 3 P. M. Frequent cloudbursts swell the river so that it is impossible to reach the field stations.

The whole party found quarters in a temple, where the photographic plates were developed and worked out. Provisions were difficult to obtain, as there was no fresh meat except chickens, and no vegetables. The muddy water from the rice fields caused ailments of the digestive organs. Nevertheless, serious troubles were prevented by the use of tinned provisions and by filtration of the water.

The most serious and unpleasant drawback was the effect of the heat and moisture on the photographic process. Work in the dark-room could be carried out successfully only during the cooler morning hours, between 2 A. M. and sunrise. The Chinese photographer was not very experienced, and worked slowly. The transport of the good and only recommendable Agfa Chromo Isolar plates was unexpectedly delayed, and the party had to start the work and make trials with many kinds of ordinary dry plates, none of which proved very successful, as some were old and all were improperly packed for the climate. There were some periods of from 6 to 10 days when the sky was clouded. Nevertheless, exposures were made in order to keep things moving. The negatives obtained, however, were so flat that the measurements were made with difficulty. After the arrival of the Agfa plates the negatives were uniformly exceedingly clear and sharp. The emulsion had a much finer grain and seemed to be more heat-resisting. From each plate several prints taken at the left standpoint were made, on which to mark out the points, etc. The new model of the stereo-comparator is equipped with a device for making these marks automatically. A pair of pictures is used for viewing in the reflecting stereoscope.

The negatives, diapositives, and prints were kept in double plate envelopes, especially made for that purpose and printed as field note books. They were made of tracing cloth, and served to protect the plates from damage. The printed record form on each envelope contains all the observations in the field and all the calculations necessary for working out the plates. It is advisable to write the field observations directly on these envelopes, in order to avoid possible confusion in transferring notes.

The theodolite at the disposal of the party was a small light instrument, with short focus (about 100 mm.) and small circle. It was not intended for preliminary railway surveys, but for explorers on general topographic work. In the course of the work it was discovered that the leveling screws were too small in proportion to the sensitiveness of the bubble tube (10″), the power of the telescope was too small (10x), the springs of the micrometer screws were too weak,

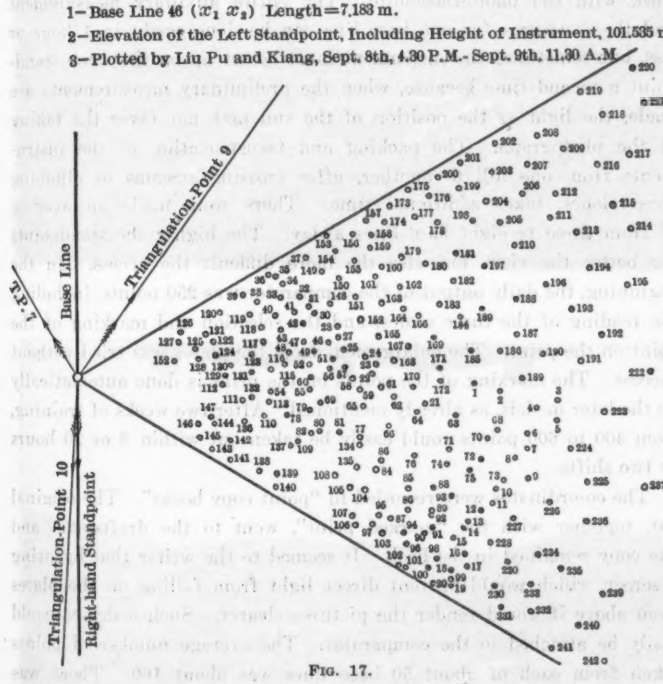


FIG. 17.

and the feet of the tripod too small for a firm setting. Also, the compass did not work satisfactorily. A desirable improvement would be a second bubble tube in the direction of the optical axis as well as a special sighting attachment, such as a direct-vision view-finder. The image on the glass plate is very dark on account of the small stop used. Therefore, the Zeiss Werkstätten proposes, as an apparatus especially suited to railway work on the scale of 1 : 5 000 to 1 : 2 500,

a photo-theodolite with $f = 190$ mm., using 13 by 18-cm. plates, with a special device for making exposures turned at 30° from the base line. In this way the image angle can be increased to 110°. In spite of the above-mentioned limitations of the instrument, the accuracy was sufficient for the short distances of the survey, as was proved by the numerous checks.

It is not easy to state the exact time required for the ordinary work with the photo-theodolite. The entire auxiliary measurement and the exposures for one base line can be done easily in 1 hour or less, but sometimes the instrument must be set up on the same standpoint a second time because, when the preliminary measurements are made, the light or the position of the sun may not favor the taking of the photograph. The packing and transportation of the instruments from one hill to another, after crossing streams or climbing steep slopes, takes additional time. There were made an average of from three to eight base lines a day. The higher the standpoints the better the view, but also the more difficult the access. In the beginning, the daily output of the comparator was 250 points, including the reading of the three scales, and the selection and marking of the point on the print. The enlargement from the plates was tried without success. The marking of the points on the print is done automatically on the later models, as already mentioned. After two weeks of training, from 400 to 600 points could easily be taken out within 8 or 10 hours by two shifts.

The co-ordinates were recorded in "point copy books". The original list, together with the "marked print", went to the draftsman, and one copy remained in the book. It seemed to the writer that by using a screen which would prevent direct light from falling on the plates from above it would render the pictures clearer. Such a device could easily be attached to the comparator. The average number of points taken from each of about 50 base lines was about 160. There was an average of one point per 25 sq. m. The area of the drawing plate was about 1 sq. m.

For the scale adopted, 1:2 500, and with a maximum distance of 1 km., a sheet 40 by 40 cm. was required.

After finishing the survey of this rocky valley, work on the less rough country was commenced, and there the conditions were a little different. It was to be mapped on the scale of 1:5 000. It was in-

Fig. 18.—Right-Hand Photograph.

Fig. 19.—Left-Hand Photograph.
Base Line 46 ($x_1 = x_2$). Length = 7.183 m.
(From Figs. 18 and 19, the Point Sheet, Fig. 17, was Plotted.)

FIG. 18.—RIGHT-HAND PHOTOGRAPH.

FIG. 19.—LEFT-HAND PHOTOGRAPH.

[illegible]

(From Figs. 18 and 19 the Plan shown Fig. 17 was Plotted.)

tended to make from all base lines further exposures with the optical axes turned at 30°, for the purpose of trying out this scale with the now more experienced staff and with the new plotting method recently published by Dr. Pulfrich. It would have been very valuable indeed to continue in this country (which also could have been surveyed tacheometrically) with the newly arrived fresh and uniform plates and during the cooler autumn weather. Unfortunately, the writer was obliged to return to Peking earlier than he intended, in order to resume his lecture work. The entire result of the work consisted of the following collection of data.

(1) Level notebooks for the bottom of the valley and photographic standpoints.
(2) Angle measurements of triangulation and trigonometric levels.
(3) Calculations of triangulation and trigonometric levels.
(4) Point co-ordinate books, with calculated elevations.
(5) "Point sheets" (tracing paper, 40 by 40 cm.) with "point prints."
(6) Supplementary tacheometric notes.
(7) Photographic material; for each base line three double envelopes containing respectively, negatives, diapositives, and prints.
(8) Finished map, and prints from it.

The whole material was collected and filed in an especially designed chest of drawers, Fig. 21, so that reference and revision could be effected readily.

Conclusions.

The result of this first trial in applying stereoscopy to preliminary railway surveys in China can be considered as satisfactory and encouraging, notwithstanding that the instruments were not quite suitable, that the staff was inexperienced, and that the climate was unfavorable. Although the territory could have been surveyed by tacheometry, it would have required seven times as long. The greatest advantage is that it is not necessary to enter the whole of the territory surveyed, and that the outdoor work takes relatively little time. Later, the working out of the plates for this survey could have been done at Hankow, where there were cool rooms, ice, and electric fans. An improvement for future work might be the use of a larger theodolite,

the greater weight of which would not be significant in the Orient where cheap coolie labor is to be obtained.

The costliness of the necessary precise photographic instruments is a relatively insignificant item of the capital required for the construction of a great railway, especially in China. The saving in capitalized interest by any acceleration of the construction more than justifies such an expenditure. For a like reason, even very expensive apparatus, such as Pulfrich's "stereo-cartograph", might be very advantageously utilized. This instrument plots the selected points automatically, through an ingenious system of levers connecting the scales of the comparator with a marking apparatus on the map.

The von Orel-Pulfrich "stereo-autograph" goes one step farther, as it enables the observer to move the balloon mark along the lines of equal elevation in the view, and at the same time plots the contour lines automatically.

Both these instruments have this great advantage, that they not only make possible the working out of "normal" stereograms, but also of those with equally turned axes, or even with convergent axes, with equal facility. This, as mentioned previously, is not possible in ordinary plotting. These convergent photographs are of special value, as they present the greatest stereoscopic field. If this latter apparatus (price 25 000 marks = \$6 250 gold) could have been used, the whole work described herein could have been done in a fraction of the time, by one man instead of four, and more accurately. In the transfer of the points from the point sheets, and in the interpolation of the contours, for example, there are sources of errors to which the plotting engine is not subject.

China is a field in which the photogrammetric method is especially suitable on account of its vast areas of mountainous country. In the North these mountains are bold and bare, in the South and central part they are wooded, but so sparsely that the surface of the ground is sufficiently visible. Moreover, the engineer is not provided by the Government with serviceable ready-made maps as in most Western countries.

The railways are beginning to spread from the seaboard across this mountainous country in all directions. The German section, from Hankow to Ichang, is still for the greater part in flat country, only the part herein described being mountainous, but the American section,

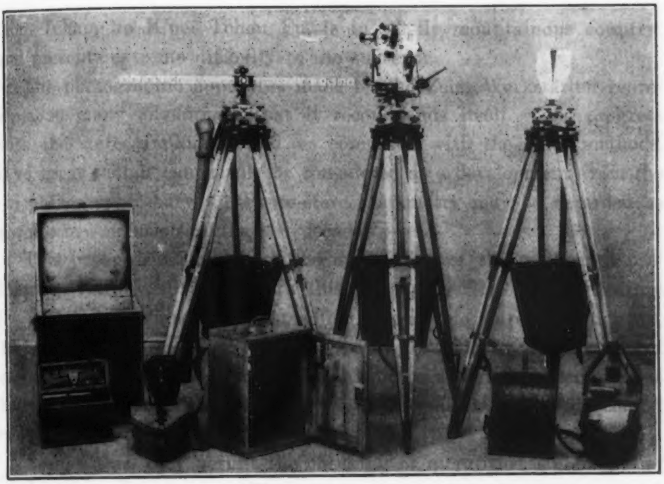


FIG. 20.—EQUIPMENT FOR A STEREO-PHOTO-GRAMMETRICAL SURVEY.

FIG. 21.—CHEST OF DRAWERS FOR STORING MATERIALS USED IN A STEREO-PHOTO-GRAMMETRICAL SURVEY.

from Ichang to K'uei Tchou Fu, is in wholly mountainous country, and presents extreme difficulty to the surveyor.

The photographic apparatus made by the Zeiss Werkstätten represents so many advantages that it recommends itself. The progress with the stereographic method, as compared with the older methods, is so great that it can hardly be considered as a development from the old photographic survey. Photo-stereogrammetry may be regarded as a quite new creation of the last decade.

This paper deals only with the application of the method to preliminary railway survey. With this its engineering possibilities are by no means exhausted. Instead of the simple unrelated pictures, 18 by 30 cm., formerly taken at intervals to indicate the progress of earthwork or structures, stereographic pictures can be taken from a base line fixed once for all. Such pictures would give a much clearer conception of the progress of work.

Besides this, they permit the checking of quantities, statements of earthwork in big cuts or fills, masonry masses, or dimensions which will later be inaccessible, and the agreement of structure to design, and this can be done quickly and accurately at headquarters. The chief engineer of a railway in China will be often unable to visit the field at the most desirable time, on account of the great distances, difficulties of travel, unfavorable climate, etc., and the demand for his presence at headquarters. In such cases the stereoscope will be of especial service, as it will answer his questions accurately. It brings country formerly quite impracticable within the range of an economic survey. The civil engineer frequently has to enter new lands, and will have to make himself familiar with this, the newest branch of surveying practice.

DISCUSSION

Mr. Lavis. F. LAVIS,* M. AM. SOC. C. E. (by letter).—This description of the application of the methods of stereoscopic surveying to railroad location is interesting from a theoretical standpoint, but there is little in the paper to show its value commercially, that is, to show that the necessary information for the location of the best line for the purpose of railroad construction through this valley was obtained by this method, at less cost than would have been possible by ordinary methods. As a matter of fact, cost is hardly mentioned except to note that one of the desirable minor instrumental aids costs about $6 000.

Apparently 10 men (engineers or engineering students) consumed most of 2 months in the necessary field work covering from 4 to 6 miles of line, though this is not quite clearly stated; if so, it hardly seems as if this method would compare favorably with those ordinarily used, namely, the typical American method, or the stadia method more generally used by European engineers.

There seems to be little in the paper to indicate the final degree of accuracy obtained. Speaking generally, it would seem that a map on a scale of 1: 2 500, or approximately 200 ft. to the inch, which is virtually an enlargement from a very much smaller scale photograph, would have a rather large probability of error. As the matter stands, it would almost seem as if the topographical map produced by this method was of approximately the same value as the topographical sheets issued by the United States Geological Survey on a scale of 1: 62 500, which are excellent for reconnaissance projections, but not for anything more detailed; perhaps, however, a further explanation by the author may throw more light on this. Of course, assuming adequate results, the true test of any survey method for the location of a railway line is the cost of the final line staked out on the ground. It must always be kept in mind that the ultimate purpose of a railroad location survey is not the making of a topographical survey or the production of a topographical map, but the location on the ground of the best line the country affords under certain given conditions, and incidentally doing the work by methods which involve the least cost. The production of a topographical map, if this is necessary or even desirable, as it almost always is, is merely incidental to the final purpose, and its extent and accuracy should be governed solely by its use for the ultimate purpose in view.

It seems to the writer that, even disregarding the first cost of the very expensive outfit, and assuming that the stereoscopic method might enable one versed in its use to produce at a reasonable cost a topo-

* New York City.

graphical map having the necessary degree of accuracy to enable a line to be properly projected on it, the cost of actually reproducing this line on the ground would be considerably more by this process than by the usual American methods.* Mr. Lavis.

It may be thought that, in using the stereoscopic method, the relation of the projected line to the triangulation points might be determined from the map and the line staked out from this, but this seems to the writer to be likely to produce results very much inferior to those obtained from the reproduction of a line quite close to a previously staked-out preliminary, inasmuch as the work involved in detailed adjustment of the line to the ground in such very rough country would be greatly increased. If the author can give any further information in regard to the final application of this method to the production of a railway line staked out on the ground, rather than the production of a topographical map, which is, of course, all this paper now does, it would be of considerable interest and add greatly to our ability to estimate the value of the new method, and judge of its practicability.

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* *Transactions*, Am. Soc. C. E., Vol. LIV (1905), p. 104; and Vol. LXV (1909), p. 105.

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Paper No. 1343

THE BURDEN WATER-WHEEL

By F. R. I. Sweeny, Esq.

Synopsis.

The south supporting pier of the old Burden water-wheel at Troy, N. Y., gave way on August 22d, 1914, and since that time there has been a general and rapid decay. In view of the fact that this wheel was the greatest in America, of a type now virtually obsolete, it seems proper at this time that its history, construction, and operation should be reviewed and placed on record in the publications of this Society. The writer was fortunate in having obtained complete measurements of the dimensions of the wheel and its setting prior to its failure, and these are given in the accompanying illustrations.

A paper* by the late Joseph P. Frizell, M. Am. Soc. C. E., entitled "The Old-Time Water-Wheels of America", unfortunately, did not mention this wheel. In the discussion of that paper, however, the desire was expressed that it be pictured, "and its efficiency, present condition and relative usefulness made known". Partly in response to this and to the feeling previously expressed, this paper is presented.

Historical.

Water-wheels, in the past, have been used for a variety of purposes, but it is doubtful if there ever existed a case having such a historic setting as the old Burden water-wheel. It is intimately associated with

* *Transactions*, Am. Soc. C. E., April, 1893, Vol. XXVIII, p. 237.

those great inventions of its designer and was born of the same genius, so that it is difficult to separate it from its setting. Thus, in order to understand the conditions leading up to its origin, it is necessary to revert to a brief historical outline prior to its inception.

Henry Burden, the designer of the wheel, was born in Dunblane, Scotland, in 1791, and after receiving an engineering education in Edinburgh, came to the United States in 1819, settling in Albany, N. Y., where he engaged in the manufacture of agricultural implements. In 1822 he moved to Troy, and took over the superintendency of the Troy Iron and Nail Factory, at that time a comparatively insignificant establishment. He began at once on his career as an inventor and manufacturer, changing the whole process of the manufacture of wrought iron and its products, and transforming the Troy Iron and Nail Factory into the largest works of the kind. It will suffice to state that it was here that he invented, built, and, for years, operated machines for the manufacture of ship and railroad spikes, nails, and horseshoes, as well as his concentric squeezer for reducing puddle balls. Thus, it will be noted that he was very progressive, constantly aiming to operate the plant more efficiently. The factory became very profitable, and because of his various inventions, Mr. Burden gradually acquired full interest, and in 1840 became proprietor. The firm was known thereafter as Henry Burden and Sons. It appears that in the early history of these works five small water-wheels were used, but, about 1838, due to the tremendous increase in the volume of business, it became necessary to secure more power with which to operate the new machines.

The foregoing were the conditions incident to the construction of the large wheel which is the subject of this paper.

Description.

The works were on the banks of the Wynants Kill, a small stream entering the Hudson River a few hundred yards below, as shown by Fig. 1. Quite close to the plant are the Burden Falls having a height of about 50 ft. and followed by a succession of rapids. The flow of this little stream was quite variable, and at some periods in the summer almost ceased entirely, being too low to furnish sufficient power. About 10 miles above this point the stream has its source in four lakes, and at the plant the total drainage area is approximately 36 sq. miles.

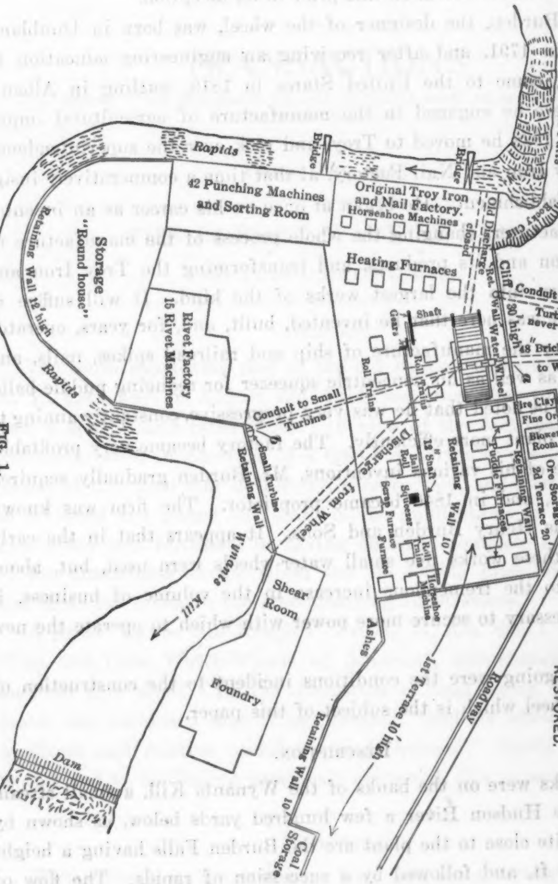


FIG. 1.

LAYOUT OF HENRY BURDEN & SONS' FACTORY, SHOWING RELATION OF WATER-WHEEL TO MACHINERY, RESTORED.

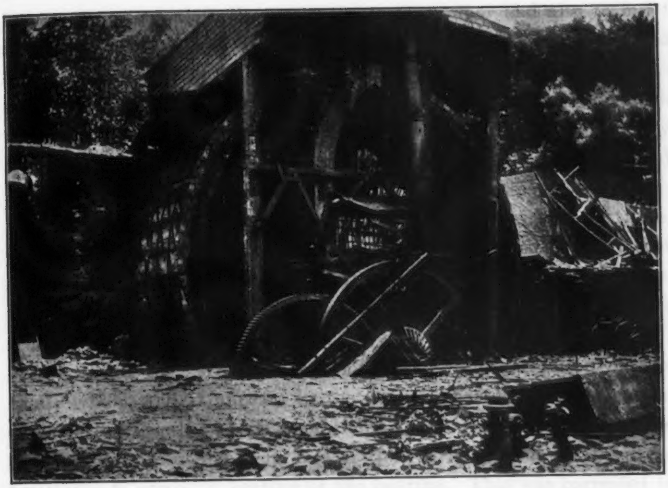


FIG. 2.—BURDEN WATER-WHEEL AFTER DISMANTLING OF WORKS, SHOWING PORTION OF ROOF.

FIG. 3.—BURDEN WATER-WHEEL, SHOWING CONDITION IN 1913.

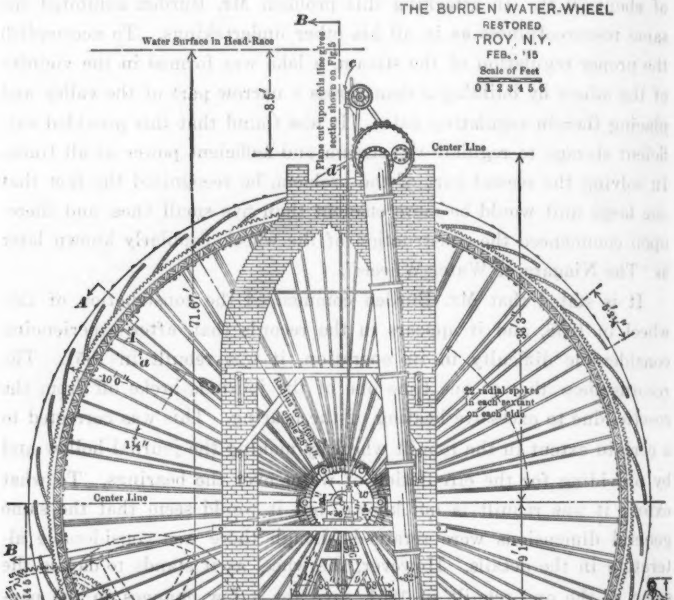
It is evident, therefore, that to accomplish the results sought it would be necessary to regulate the stream in some manner and also to build a wheel of sufficient size to utilize practically the total available head of about 70 ft. In attacking this problem Mr. Burden exhibited the same resourcefulness as in all his other undertakings. To accomplish the proper regulation of the stream, a lake was formed in the vicinity of the others by building a dam across a narrow part of the valley and placing therein regulating gates. It was found that this provided sufficient storage to regulate the stream and sufficient power at all times. In solving the second part of the problem, he recognized the fact that one large unit would be more efficient than five small ones, and thereupon commenced the construction of the wheel, popularly known later as "The Niagara of Water-Wheels".

It is stated that Mr. Burden commenced the construction of this wheel in 1838, and it appears in the records that, after experiencing considerable difficulty in its operation, it was rebuilt in 1851. The records show that at one time one of the journals broke off from the rosette, due to excessive heating of the bearing. This was corrected to a certain extent in the rebuilt wheel by making the journal hollow and by providing for the circulation of water over the bearings. To what extent it was rebuilt is not known, but it would seem that the same general dimensions were retained, though there was considerable alteration in the details. However, the wheel as it stands to-day is the same as the one rebuilt in 1851, save for a few changes in the penstock, which was constructed of wooden staves instead of the riveted iron pipe now standing. It is stated that, outside of the ordinary repairs incident to the operation of the wheel, the only part that was renewed was the segment gearing, which was replaced about 1882.

The dam was a few hundred yards above the falls, and the water flowed in a canal along the hillside, ending at a point about 100 ft. from the center of the wheel, as shown by Fig. 1. From this point it was carried in a brick conduit, 4 ft. in diameter, to the edge of the cliff, and there this conduit was joined to the riveted steel penstock, also shown by Fig. 1. In the canal at the head of the conduit there were head-gates.

The wheel is of the overshot type, and in general consists of a central hub to which are fastened radially 264 iron rods, 1½ in. in diameter. These are fastened to 10 by 10-in. Georgia pine timbers

forming the soling on which the floats or buckets are built. On each side of the wheel on the soling is fastened the segment gearing which gears with two pinions keyed to the same shaft, as shown at the

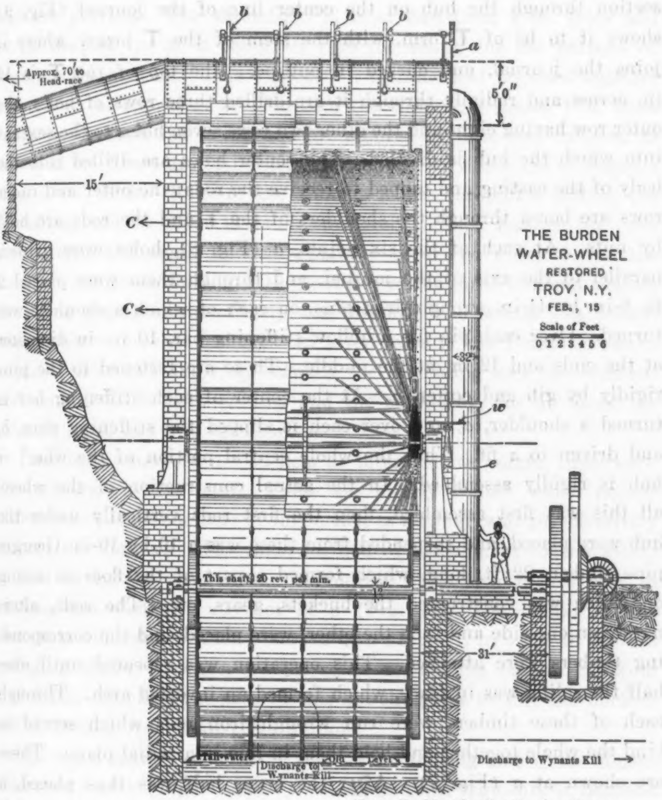


FRONT ELEVATION

Fig. 4.

ground level in Figs. 4 and 5. The wheel stands in a pit, about 20 ft. deep at the lowest point, cut out of solid Hudson River shale. The sides of the pit, along the faces of the wheel, are lined with brick to the ground level. At the center, under the bearings, the wall is

carried up about 4 ft. on the north side and 9 ft. on the south. On each of the piers there is a cast-iron base-plate on which rests a cast-iron pedestal, shown in Figs. 4, 5, and 8. The reason for the shorter pedestal on the south side is apparently due to the fact that the



Note: See Fig. 4 for line showing plane on which this section is taken—Line *B-B*.

SIDE ELEVATION AND PART SECTION.

FIG. 5.

rock on that side is against the face of the cliff. Before the wheel was constructed, the cliff extended out for a considerable distance, but this was all removed in order to obtain the proper setting for the wheel.

On the pedestals rest the main bearings which support the wheel. The hub is made up of two rosettes fastened together by hollow bars, as shown in Figs. 8 and 9. Each of these rosettes is 7 ft. in diameter, and is cast integral with the journal, which is 20 in. in diameter. A section through the hub on the center line of the journal (Fig. 9) shows it to be of T-form, with the stem of the T larger where it joins the journal, and curved in outline. The top of the T is 12 in. across and radially through it are drilled three rows of holes, the outer row having eight and the other two rows seven holes to the sextant into which the hub is divided. The center holes are drilled into the body of the casting and tapped to receive the rods; the outer and inner rows are bored through the shoulders of the T and the rods are held by nuts. At each of the six points, *a* (Fig. 8), holes were drilled, parallel to the axis of the journal, and through them were placed 2 ft. 6-in. by 4½-in. wrought-iron pins, on each of which a shoulder was turned. Over each pin fits a hollow stiffening bar, 10 in. in diameter at the ends and 12 in. at the middle. These are fastened to the pins rigidly by gib and cotter, *c*. At the center of each stiffening bar is turned a shoulder, *d*, and over each is slipped the stiffening ring, *b*, and driven to a fit. Thus the whole central portion of the wheel or hub is rigidly assembled. In the actual construction of the wheel, all this was first assembled, then the first rods vertically under the hub were placed, and suspended from these was a 10 by 10-in. Georgia pine timber, 22 ft. long, which formed a part of the floor or soling on which was constructed the buckets, gears, etc. The rods, alternately on one side and then the other, were placed, and the corresponding timbers were attached. This operation was repeated until one-half the soling was in place, which formed an inverted arch. Through each of these timbers were run wrought-iron rods which served to bind the whole together and hold them in line in a radial plane. These are shown at *a* (Fig. 7). After the lower half was thus placed, a center was built for the upper half, as would be done for a full centered arch, on which the remaining portion of the soling was placed.

It will be noticed that the center row of holes in the rosettes requires that a turnbuckle be placed on each rod running to them, otherwise there would be no way to regulate their tension. The rods passing through the outer and inner rows of holes, which are bored through the shoulders of the T, permit of adjustment with nuts, as shown at

e (Fig. 9). In order to stay the wheel against any axial motion, the three rows of rods on each side are disposed as shown in Figs. 5 and 7, which is the same as in a bicycle wheel.

Figs. 6 and 7 show the method of constructing the buckets and segment gearing. The shrouding, *b*, is of cast iron, $\frac{3}{8}$ in. thick and 29 in. deep, 2 in. of it being set into the soling for the purpose of caulking the buckets. This was cast in sections and fastened together with rivets as shown at *c*. Through the shrouding passes 1-in. bolts which go through the soling and are secured by nuts and washers, as shown at *d*. There are 36 buckets, the timbers of which are of Georgia pine, 2 in. thick. Each of the timbers passes through a slot in the shrouding, and in order to caulk effectively against leakage the ends were kerfed and small wooden wedges were driven which forced the wood against the shrouding. On the inside of the shrouding and against the soling a quarter round strip was fastened, and oakum was packed behind this to prevent leakage. In after years it was found necessary to place the small angle, *e*, between the shrouding and the segment gear and to caulk between this and the shrouding, for at times the water squirted out into the shop.

In order to support the buckets at intermediate points, three cast-iron spacers, *f*, were used. These are merely frames which permit of free access from one part of the bucket to another, and yet effectively support the timbers. The same method of attachment to the soling is used as in the case of the shrouding. The spacers are cast in sections and fastened together at *g* (Fig. 7). The planks of the buckets are doweled to each other, as at *g* (Fig. 7), in order to prevent springing under pressure, which would render the oakum caulking ineffective. Another function of this doweling was to stay the buckets and shrouding from transverse motion.

The main gear is made up in segments 6 ft. long and of the dimensions shown at *h* (Figs. 6 and 7). Each of these segments is fastened to the soling by five bolts which pass clear through it and also through a $\frac{3}{4}$ by $5\frac{3}{4}$-in. bar inside the wheel. There is one of these gears on each side of the wheel, as shown by Fig. 7, and they gear with two pinions, 7 ft. in diameter and 9-in. face. These pinions are keyed to a shaft which is 12 in. in diameter and is supported on bearings on each side of the pit. The shaft extends out into the room about 31 ft. from the face of the wheel, and terminates in a large gear 16 ft.

e (Fig. 5). In order to stay the wheel against any end motion the three rows of rods on each side are dished as shown in Figs. 6 and 7, which is the same as in a bicycle wheel.

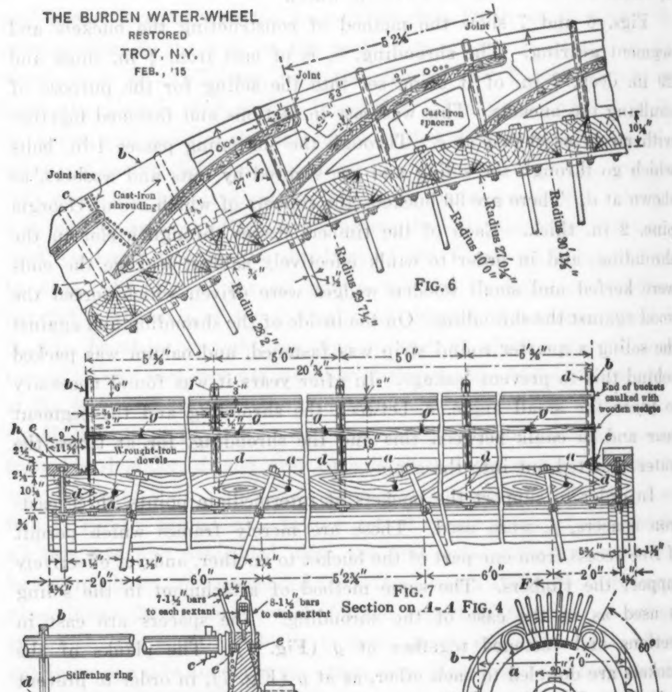


FIG. 6

FIG. 7

Front view of pedestal, bearing, etc.
FIG. 8

Side view of pedestal, rosette and stiffeners. Section on F-F
FIG. 9

each side of the wheel, as shown by Fig. 7, and they gear with two pinions, 7 ft. in diameter and 2-in. face. These pinions are keyed to a shaft which is 19 in. in diameter and is supported on bearings on each side of the pit. The shaft extends out into the room about 21 ft. from the face of the wheel, and terminates in a large gear 16 ft.

in diameter. This in turn meshes with a 44-in. pinion which is keyed to a shaft carrying a fly-wheel 20 ft. in diameter. On the same shaft and adjacent to the fly-wheel is keyed a miter gear which meshes with another keyed to the line shaft. This line shaft ran the length of the shop, a little above ground level, and was 12 in. in diameter.

It is seen from the drawings that, the power being taken off at the periphery of the wheel, instead of at the axis, the only torque produced at the axis was due to journal friction. It is estimated that the wheel weighs about 250 tons in a soaked condition or 125 tons on each bearing. This weight was sufficient to require some means of taking care of the frictional torque produced, and accordingly the two rods, *a* (Fig. 4), were placed and provided with turnbuckles.

A very novel feature is the method of bringing the water to the buckets. The canal ends at a point about 100 ft. from the center of the wheel, and there the water is taken into a brick conduit through a head-gate. This conduit is made of three rings of brick, each ring being cemented and hooped around with wrought-iron bands. This stands to-day in apparently perfect condition. At the edge of the cliff this brick conduit is joined to a riveted iron penstock which rises and is carried over the top of the wheel, resting on brick arches close to each side. All this has fallen in, due to the failure of the foundation walls on August 22d, 1914. Up to that time the wheel was in almost as good condition as when it stopped in 1896. On the top of the iron penstock there are five iron brackets supporting a shaft which terminates in a worm wheel and has keyed to it four pinions in intermediate positions, as shown at *b* (Fig. 5). In the side of the penstock there are four rectangular orifices, equally spaced over the breadth of the wheel. Inside of each there is a sliding gate which is fastened to a stem passing up through a stuffing-box and connected to a rack. This rack gears with the pinion on the shaft referred to, and is held in gear with it by an idler which runs on a small shaft supported by the brackets. At *c* (Fig. 4) a worm engages the worm wheel and is keyed to the shaft, *d*, which runs down to the hand-wheel, *w*. This is supported by the bracket, *e* (Fig. 5), which is attached to the pedestal of the main bearing. In the operation of the wheel, a man was constantly kept at this hand-wheel and, at a signal from the shop, would admit more or less water and thereby maintain the speed constant at 2½ rev. per min.

The discharge from the wheel was taken through a tunnel cut in the solid rock to the creek, a distance of about 200 ft. This represents the normal tail-race; but, in the early history of the wheel, a suit was brought by the owners of the property on the other side of the creek and adjoining the falls, demanding that the water be returned to the stream at the foot of the falls. Due to the numerous small rapids between this point and the point of normal discharge, this meant that the bottom of the wheel would be submerged for 2 or 3 ft., which would cut down the power very materially. This matter was adjusted subsequently, and the old discharge was opened again. The two discharge tunnels are shown on Fig. 1; the short one to the foot of the falls was closed after the adjustment of the difficulty. The vertical cast-iron pipe bolted to the end of the penstock and running down beside the pedestal was used as a penstock to a small turbine at *b* (Fig. 1). The conditions that brought about this addition are not known.

The drawings only show the restored wheel. A part of the roofing is shown on Fig. 2. The point of connection with the main factory roof is clearly shown. The wheel stood in the main room of the works (Fig. 1), and being under the same roof was afforded protection from severe weather. The rear wall of the works was built up on the edge of the cliff immediately adjacent to the wheel, and for a height of 10 ft., and the roof started from this. In this wall, and opposite the wheel, a door was placed, and from this a gangway was constructed up alongside the penstock and over the top of the brick arches which supported it. The wheel, just after it was built, became quite famous, and was constantly inspected by a large number of visitors. The gangway leading to the top was opened to them by the proprietor, and quite a good idea of the magnitude of the wheel could be obtained by looking down into the shop from this height.

In severe winter weather, after a shut down, considerable trouble was caused by ice, which formed on the top and had to be chopped away before the wheel could be operated. Of course this never occurred during normal operation, as it was prevented by the heat from the furnaces. The bearings were lubricated with suet, and the caps, being hollow, were packed with this lubricant. However, the bearings became heated occasionally, and, to remedy this, water was circulated over them. The wheel was very rigid for a machine of this size, yet there was some vibration, and this eventually loosened some of the rods

and occasionally broke one. The proper tension had to be maintained at all times, in order to secure the necessary rigidity and also make the gears mesh properly. It was also the constant aim of the attendants to maintain the wheel in a uniformly soaked condition because any drying out would tend toward an unbalanced operation. This also would affect the proper meshing of the gears.

Fig. 1 gives a good idea of the general layout of the plant. In operation the following machines were driven by the wheel: one rotary concentric squeezer and muck-bar train, six 9-in. trains for rolling horseshoe and rivet iron, six horseshoe machines, fifteen rivet machines, forty-two punching machines, machine shop, roll lathes, shears, and other machines required in a rolling mill.

The wheel was in continuous operation for a period of 45 years, often running day and night for long periods; but, due to the growth of the business and the necessity for more power, the works were abandoned in 1896 and the machinery was moved to the steam mill which is a short distance away on the banks of the Hudson.

The wheel is rapidly falling into decay, and before long its destruction will be complete. The supporting piers have failed, and the wheel has settled into the pit. The brick arches also have fallen, and the iron penstock is resting on the top of the wheel, broken away from its connection with the brick conduit and vertical cast-iron pipe. As one stands and looks on this, he is forcibly reminded of the irresistible advancement in engineering lines, there is such a contrast between this huge motor and the compact, high-power units of to-day.

Power and Efficiency.

Thus far nothing has been said of the power and efficiency of this wheel, but, as this is of interest, the writer has worked up, from known conditions of operation, the curves shown. A brief explanation of the method of deriving these curves is given.

To determine the power and efficiency of an overshot wheel, for different conditions, by the methods given by Weisbach, would be very troublesome, and so the writer has resorted to a graphical solution. It will be assumed at the outset that each bucket is completely filled at the top of the wheel, and then the intermediate conditions can be readily determined.

It is clear that if the bucket is initially completely filled, it will start spilling at once and continue to do so until it is completely discharged. To determine the power developed under these conditions, an integral expression of the form,

$$\text{Horse power} = \int \frac{W\,dH}{33\ 000},$$

must be evaluated. In this expression there are two variables, and, from a well-known principle of the calculus, if for all values of H we can determine the corresponding values of W, it can be evaluated. It is evident that, for every position of the bucket, the water contained therein is a definite quantity which fulfills the foregoing conditions. By making a drawing of one of the buckets to a large scale, and considering it in ten consecutive positions from the top around to the lowest point, would mean that by drawing lines tangent to the lip of the bucket every 10°, the area within each and the outline of the bucket is a measure of the quantity of water contained. The center of gravity of each section thus determined, was plotted, and the head above tail-water measured. The integral was changed to the form,

$$\text{Horse power} = \int \frac{C\,A}{33\ 000}\,dH,$$

in which C is a constant and A is the cross-sectional area of the water in the bucket. In Fig. 11, the term $\frac{C\,A^{*}}{33\ 000}$ was plotted as abscissas against H as ordinates.

This takes care of the water from the topmost bucket to the tail-race level and in order to include the effect due to the impulse of the entering jets from the four orifices the following expression was used:

$$\text{Effective head on top bucket} = (V \cos.\ O - u)\,\frac{u}{g},$$

in which V is the absolute velocity of the entering water, O is the angle the entering water makes with the tangent to the bucket, and u is the peripheral velocity of the center of gravity of the water in the bucket. V was determined by the Bernoulli equation, and found sensibly constant for different rates of discharge, because the area of the orifices is small compared with that of the conduit. By consulting the curve, Fig. 10, it will be seen that by dividing the greatest abscissa

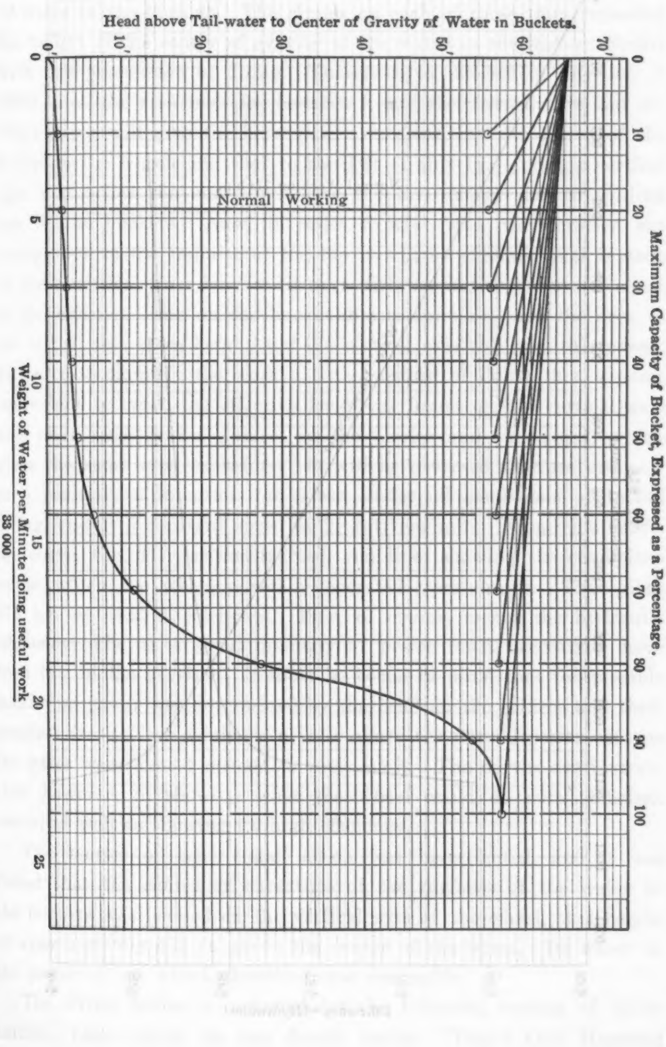

Head above Tail-water to Center of Gravity of Water in Buckets.
0′ 10′ 20′ 30′ 40′ 50′ 60′ 70′
Maximum Capacity of Bucket, Expressed as a Percentage.
0 10 20 30 40 50 60 70 80 90 100
Normal Working
Weight of Water per Minute doing useful work / 33 000
0 5 10 15 20 25

FIG. 10.

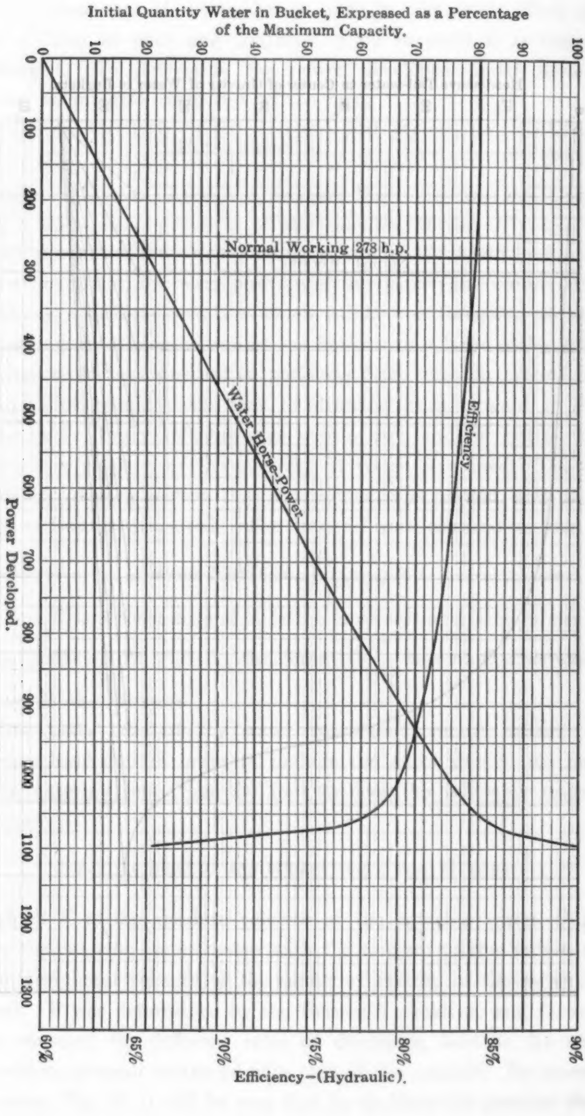

Initial Quantity Water in Bucket, Expressed as a Percentage of the Maximum Capacity.
0
10
20
30
40
50
60
70
80
90
100
Normal Working 278 h.p.
Water Horse-Power
Efficiency
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
Power Developed.
60%
65%
70%
75%
80%
85%
90%
Efficiency—(Hydraulic).

FIG. 11.

into ten equal parts by vertical lines, where these intersect the curve will give the height at which spilling commences for that quantity of water in the buckets. The circles on each of these lines represent the height of the center of gravity of the water in the highest bucket with this percentage of filling. Therefore, to the left of any one of these lines, the area included between it and the vertical axis and the lower curve will give the horse-power developed by the action of the water on the wheel, and that to the left of this line and the vertical axis and below the curve will give the lost energy due to spilling out of the buckets. Now, in order to take into consideration the energy due to the impulse at the top it will be observed that if each of these vertical lines be carried up a distance above the circles equal to the effective head on the bucket due to impulse, then the area to the left of this extra height and the vertical axis will give this power. In order to simplify the work a little, instead of doing this, a point equivalent to twice the effective head was found on the vertical axis and lines were drawn connecting the circles and this point, which gives the same area as before. It will be observed that we are now in a position to integrate for power under all conditions of initial filling, and at a constant speed of 2½ rev. per min. From this curve, the curve, Fig. 11, was constructed, and lines showing the conditions under which the wheel normally operated were drawn. This gives 278 h.p. at 84.25% efficiency. This, of course, means the hydraulic efficiency only, as a great quantity of power must necessarily have been lost in the gearing. It must be borne in mind that the possible maximum power was determined by the head on the orifices and their greatest area. This would be a little above the normal power because the gates were almost opened to their limit. The curves are instructive, however, in showing what the wheel could do with sufficient water, as well as showing its high efficiency.

The matter of centrifugal action was investigated, and it was found that the center of curvature of the surfaces of the water in the buckets was located on the vertical axis of the wheel, at a height of approximately 475 ft. above the center of the wheel. Its effect on the power of the wheel, therefore, was negligible.

The writer wishes to acknowledge the following sources of information, from which he has drawn freely: "Troy's One Hundred

Years", "Henry Burden", by Margaret Burden Proudfit; and graduating theses at the Rensselaer Polytechnic Institute by Frederick Grinnell, '55, George F. Kirby, '57, Charles McMillan, '60, Horace Crosby, '62, and Abraham B. Cox, '67. Much valuable material was obtained from these theses, as they were written while the wheel was yet new. The writer is also indebted to Messrs. J. A. Leskie and R. S. McEwan, both of whom were workmen at the old plant.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

TRANSACTIONS

Paper No. 1344

DEPRECIATION AS AN ELEMENT FOR CONSIDERATION IN THE APPRAISAL OF PUBLIC SERVICE PROPERTIES

By C. E. Grunsky, M. Am. Soc. C. E.

With Discussion by Messrs. James D. Mortimer, Joseph Mayer, F. W. Green, Richard J. McCarty, Philip W. Henry, Clinton S. Burns, Halbert P. Gillette, W. Kiersted, J. N. Dodd, William B. Bosley, and C. E. Grunsky.

Synopsis.

The object of this paper is to bring nearer to standardization the methods of dealing with depreciation when valuations are made for rate-fixing purposes; to show that the repayment to the owner of invested capital has nothing to do with depreciation; to show that the investment should not be returned to the owner unless he is to be bought out; to show that a public utility taken as a whole may be regarded as having unlimited life; to show that in fixing rates the replacement requirement may with advantage be allowed to take the place of the depreciation allowance; to show that departure of actual from probable life should be taken into account in estimating replacement requirements or depreciation; and to show that the method here called the Unlimited-Life Method of making appraisals is better suited than other methods, to serve as the basis for estimating necessary earnings.

All methods of valuing public service properties for rate-fixing purposes—to be fair to both owner and rate-payer—should be in continuous application from the beginning of operation.

It follows from this that the past history of expenditures and earnings should be given consideration and that it may be unfair to assume that, because of depreciation in fact, a part of the investment has been amortized and that if not amortized the fault is the owner's. (Knoxville case.)

The invested capital should remain unimpaired. The owner's earnings should include whatever may be necessary to meet maintenance, repairs, and replacements, but no part of his investment should be returned to him, unless the acquisition of the property is involved.

The situation is not altered even in those cases in which the owner diverts to outside uses the funds which he has earned to meet replacements. The owner's obligation to replace the worn-out parts remains, and must be met if the rate-payer is to be served satisfactorily.

Amortization of the invested capital becomes necessary only when the public service property is to be acquired by the public.

The allowance in the earnings which is usually made to the owner under the name of depreciation, whether applied to this use or not, is really intended as an allowance to meet replacements.

Replacement requirements can be determined from the cost to replace and the time when the replacement will probably take place.

The time when an article which has already been in use for some time—which, in other words, has acquired age—will go out of service cannot be determined by subtracting the age from the probable life of a new similar article. It is dependent on the condition of the article. It is approximated from the article's expectancy—from its remaining probable life. Expectancy changes with age.

When expectancy is given due weight, the application of the Equal-Annual-Payment and other similar methods of valuation becomes difficult if not impracticable.

Expectancy is capable of much closer approximation than by the subtraction of age from probable life.

Taken as a whole, each public service property may be regarded as having unlimited life. The probable life of its parts—whether only a railroad tie or an electric generator—does not alter this unlimited life

of the whole. The expectancy of each part, however, may be used in estimating the "owner's obligation to replace", which is another name for what is usually called "accrued depreciation".

The obligation to replace each part, expressed in dollars, grows with the age of the part. If the replacement requirement is properly estimated and covered by the earnings, the owner can keep the plant at 100% efficiency, based on the standard established by the plant itself; but such replacement allowance cannot be regarded as a return of capital until the time comes when the owner is relieved of any further obligation to replace.

The rate of depreciation is an aid in making an advance determination of the replacement requirements, and to this extent is an element to be considered in fixing rates.

The accrued depreciation, or the accrued obligation to replace discarded or inefficient articles, becomes of interest when for any reason the remaining value of physical elements is to be ascertained.

Conclusion.—The Courts are in error when they hold it to be always necessary to give depreciated value consideration when rates are to be fixed.

The fair application of any method of valuation to a property which has acquired age, when this valuation is to be used as a basis for fixing service rates, involves due consideration of past history, particularly in so far as the method is concerned by which necessary earnings in the past were estimated.

Continuity of application is essential to the fairness of the various methods of making appraisals for rate-fixing purposes.

No part of the capital properly invested in a public service property should be returned to the owner until a transfer of ownership to the public is contemplated.

The appraiser should concern himself with the replacement requirements, rather than with amortization of capital or with depreciation.

In estimating depreciation, accrued depreciation, or replacement requirements, the expectancy, properly determined, should be substituted for probable life new less age.

All public service properties should be regarded as having an unlimited life, and, if thus regarded, a new method of valuation becomes possible in which operating expenses are made to include the

replacement requirements and under which there will be no amortization of capital.

The Unlimited-Life Method of valuing public service properties has the great advantage of requiring the smallest earnings in the early years of the property's life; it is simpler than all others in its application; and it is fair to both owner and rate-payer.

Despite the tendency which is apparent on the part of the Courts and the public service commissions, and especially on the part of the lawyers and experts who represent the rate-payers, to favor appraisals based on cost of reproduction, less depreciation, as a vital element to be considered in fixing rates, there is no doubt that there will be a backward swing of the pendulum, and that the fundamental principle will ultimately be recognized, under which the owner of a public service property will be regarded substantially as the agent of the rate-payer; and, as such, will be allowed, and may even be required, to keep his investment in the property unimpaired and undiminished, just as would be the case if the rate-payers themselves, through the agency of a municipality or otherwise, had constructed the public service plant. This is good policy, sound in every way, and, when well understood, being absolutely fair to all concerned, providing as it does for the most reasonable apportionment of earnings to various periods of a plant's life, should, in all cases in which it is applicable, prove acceptable alike to the rate-payer and to the owner.

Some of the public service commissions of the country, as well as the Courts, are confounding "amortization" with "depreciation". The result is unfortunate, and has led valuation experts to adopt all sorts of subterfuges to secure for their clients that equity to which, whether rate-payers or owners, they are entitled.

It has become common practice, nevertheless, frequently an improper practice, to regard as a repayment of capital all so-called "depreciation" which has been allowed in the earnings (and, let it be assumed, which has actually been collected from the rate-payer).

Any repayment of capital is "amortization", and has nothing whatever to do with the depreciating service value of any part of a public service plant.

The repayment of capital (amortization), which it has become customary to assume is taking place when earnings include an allow-

ance for depreciation, is an unfair burden on the rate-payer in the early years of operation.

This can be and is avoided under the rational system of making suitable provision for the replacement of worn-out parts of the property, instead of endeavoring to amortize the investment.

To make this clear and to point out some of the errors into which Courts and some experts are drifting is the main purpose of this paper. Incidentally, a fact which is too often lost sight of will be emphasized, namely, that the various methods of computing the current depreciation allowance which should be covered by the earnings are only then correct when applied consistently and continuously throughout the probable, or, as the case may be, the useful, life of an article; that, therefore, the dictum of the Courts, according to which no regard should be paid to the fact that past earnings may have been insufficient to provide a depreciation fund, is at fault, and should be suitably modified or at least qualified.

The writer will endeavor, moreover, to show that appraisals for rate-fixing purposes may always be made at the full amount of the investment and that, when applicable, the method of computing and allowing replacement requirements under the assumption that the property as a whole has unlimited life, instead of allowing depreciation as ordinarily defined, is the one which should find general acceptance.

Fair Value.—The term "fair value" as used in connection with rate-fixing is difficult to define. This term is used in a measure, no doubt, for the purpose of ruling out "book value" or actual investment, which may include items and amounts of doubtful propriety, and to rule out values based on stock and bond issues and the market values of such securities. The term has not yet been defined satisfactorily, and no attempt will be made here to reconcile divergent views in relation thereto; but attention may be called to the difficulty which has been experienced by all who have attempted to make appraisals for rate-fixing purposes, in reconciling "fair value" to a purchaser with the "fair value" which is to be considered when fixing rates. Why should there be one valuation for purchase and another for rate-fixing? The answer that has been given by the Courts (Knoxville case) is practically to the effect that there should be no such difference, and experts have been put to it to determine values so that any such difference shall disappear.

The value to an investor is unhesitatingly determined from the net earnings, with due regard to hazards of the business. The value for rate-fixing purposes is to be that value on which, with the same regard for the hazards of the business, the owner is to be allowed to earn a fair interest return. Value should be the same whether determined by a rate-fixing body or whether determined for a purchaser. "Fair Value", then, is necessarily based on proper and reasonable investment which may be ascertained from cost records, and, when cost records are not available, is usually estimated by the "cost-of-reproduction" method. Any determination of value by comparison with the cost of the next best substitute for the established utility is at times unwarranted and absurd, and has no place in determining the equities between the owner and the public whom he is serving, beyond establishing an upper limit of possible value.

An appraisal of "Fair Value", in this sense, may include certain more or less indeterminate elements, such as the cost of establishing the business, including early deficient earnings; but no definite or precise rules can be laid down for determining these, as it would be eminently unfair to reward the least successful enterprise which shows greatest early losses and highest unproductive expenditures, by granting it the greatest allowance for so-called "going concern" or "establishment of business" or other similar values.

"Investment" or "Capital Invested."—This is the aggregate of the reasonable and proper expenditures which have been made to render the property in question efficient for the purpose for which it is intended.

"Depreciation."—This is the lessening in worth of any perishable article or property. It is not, therefore, to be measured by inherent deterioration, due to wear and tear, but solely by a comparison between the probable life, the expectancy, and the cost of replacement at the expiration of the term of the article's actual serviceability.

"Annual Depreciation" or the "Annual Depreciation Increment."—This is the annual theoretical lessening in worth, expressed in dollars.

"Accrued Depreciation."—This is the difference between the original cost and the "remaining value".

"Remaining Value" or "Present Value."—This is the present worth of any article, dependent on, and to be computed from, the three elements: (*a*) the probable useful life of the article new; (*b*) its probable remaining life; and (*c*) the cost of replacement, which, when prices

are not subject to change, is the original cost less the residual value of the article when it goes out of use.

"Residual Value."—This is the value which remains in any article after it has ceased to be useful as an integral part of the public service property. It may be scrap value, or it may be the price at which it is disposed of for use in connection with some other property. It is usually estimated as the value to an outside purchaser, less the cost of delivery to such purchaser.

"Unrecovered Depreciation" or "Uncollected Replacement Annuities."—This refers to the allowed depreciation increment or to the allowed replacement annuity which the earnings are inadequate to cover, and which, in consequence thereof, could not be collected.

"Probable Life."—This is the probable time, usually expressed in full years, during which any article will render efficient service. This is to be determined from the sum of experiences with like articles and with due regard to local conditions. It depends not alone on the time required for an article to deteriorate and become valueless by use, by ordinary wear and tear, but also on the time when, by reason of accidental destruction, inadequacy, or obsolescence, the article must be replaced by a new one better adapted to fulfill its purpose.

"Remaining Life" or "Expectancy."—This is the probable time, expressed in years, during which any article which is no longer new will continue to render efficient service. The remaining life cannot always be determined from the probable life of the article when new by subtracting therefrom its age. The condition of the article and the local circumstances should be taken into account in estimating the remaining life.

A high-duty pump, which originally had a probable life of 25 years, when it reaches the end of this term, may be in a condition almost as good as new. It has escaped the possible accidents of the early years of its life, and by careful attention and replacements of its wearing parts is still rendering first-class service. The value of this pump is not to be written off the books, neither should it be regarded as good as new. Its value is ascertained by determining its probable additional years of usefulness and the probable cost of replacing it at the end of the new term.

An irrigation canal usually improves with age. So far as wear and tear is concerned, it has unlimited life; but, under the develop-

ment of extensive areas, the small original canal may in the course of time be superseded. The probable life of this canal, and therefore, too, the annual replacement increment, is estimated on some assumption relating to the rate of this development. Finally the time comes when the project for a comprehensive canal system has taken definite shape and it may reasonably be assumed that within a definite period, five or ten, or some other number of years, the original canal will be superseded; its diverting dam and its head-works, and perhaps the canal itself, will then be abandoned. The remaining life or expectancy of the canal is at that time only five or ten, or some other number of years, as the case may be, and within this time the remaining investment in or the value of the canal is the amount under consideration for replacement.

"Amortization."—In the sense in which this term is used in this paper, it applies to the retirement of capital. Ordinarily, the discarded article is replaced by a new one which will render equivalent or better service. The capital to be amortized during the life of the article is, for all practical purposes, its replacement cost, being the cost of installing an equivalent new article, less the residual value of the original article.

"Annual Replacement Increment" or "Replacement Annuity".—These are terms used to designate an annual allowance or an annual contribution to a replacement fund, such that this annual allowance, together with interest thereon (at the rate of the net earnings of the property) will, during the life of any perishable article, amount to its replacement cost.

"The Obligation to Replace."—This is an obligation, carried by every owner of public service property, to replace articles forming essential parts of public service plants when worn out or no longer capable of rendering adequate service. The "Obligation to Replace", when expressed in dollars, is equivalent to the accrued theoretical depreciation or to the sum which should have been earned and accumulated to meet necessary replacements. This obligation to replace, together with deferred maintenance, must always be considered by the appraiser of property which is to be purchased. The investment less this obligation to replace (also less deferred maintenance) is, in other words, a measure of present worth.

"Deferred Maintenance."—This is the neglect, expressed in dollars, which has resulted from failure to keep an article in good condition and repair. It is the sum which should at once be expended to restore the article to ordinary good service condition, and to protect it in such a way against causes of rapid destruction that its deterioration will not be unduly rapid. Ordinarily, there should be no "deferred maintenance". By proper attention to maintenance and repairs, the service rendered should be up to the standard which is expected of the plant. In other words, the plant should, practically at all times, be at 100% efficiency.

"Wear and Tear."—This is the term applied to the deterioration of an article from use. The article is kept in serviceable condition by the renewal of its worn-out parts and by suitable attention to its wearing parts. Maintenance and repair are to be included in operating expenses, and are not covered by depreciation in the sense in which used in this paper. The expenditures for repairs and maintenance are intended to keep the article at or near 100% efficiency. Repair and maintenance expenses usually increase somewhat with the age of an article, and, when this increase is worthy of note, may be an element to be taken into account in determining present worth.

"Replacement Cost."—This is the cost of the article with which a worn-out article is replaced, less the residual value of the original article. It is the cost of effecting a change from the worn-out part to a new part of equal service value. This is sometimes called "wearing value".

"The Sinking-Fund Method."—This method of making appraisals and of determining necessary earnings is one under which the annual allowance for depreciation is uniform in amount. This depreciation allowance is the annuity which, together with interest at the rate of the net earnings of the property, will, during the probable life of an article, amount to its replacement cost. The replacement, it is assumed, is to be accomplished at the end of the probable life of that article. There is no repayment of capital. The investment remains undiminished. This, therefore, is a 100% valuation method and might be thus designated. It is the method referred to by the writer as Method No. 1, in a former paper* on appraisals.

* "The Appraisal of Public Service Properties as a Basis for the Regulation of Rates," *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 828.

The "Equal-Annual-Payment Method", as described in the report of the Special Committee on Valuation, as presented to the Annual Meeting of January 21st, 1914, refers to that method which makes the annual depreciation, or amortization increment, an amount increasing from year to year. The annual depreciation is estimated by sinking-fund methods. It is, for any year, equal to the annuity which will retire the remaining value in the remnant of the original probable life term and, uniformity of interest rates being assumed, it will, when added to the interest on the remaining value, always amount to the same sum. The application of this method is exactly equivalent to the application of the Sinking-Fund Method.* When depreciation allowances computed under this method are actually earned from the time an article goes into service. they may be considered as refunds of invested capital. The invested capital, in that case, decreases as depreciation, if thus earned, accrues. Depreciation earnings may properly be regarded as being thus applied to retire capital, but this method of estimating what should thus be earned from year to year is undesirable, because it involves frequent, elaborate, and cumbersome re-estimates of remaining value, requiring the services of experts, although it is absolutely identical in results with other methods which have the advantage of simplicity. Furthermore, when not applied from the beginning, there may never be any surplus in such an amortization account for application to a refund of early investments and this portion of the investment may remain undiminished.

"The Straight-Line Method."—This "method of estimating the annual depreciation", or "the annual amortization installment", makes the annual amortization of capital uniform throughout the probable life of an article. The annual amortization installment is estimated from the original cost of an article and its probable life, by dividing the cost by the number of years. This, like any other amortization method, is justified on the theory of the immediate application of an annual installment to the retirement of capital. In this respect there is an essential difference between this and the Sinking-Fund Method, under which no capital is retired, and also between this and the Unlimited-Life Method, under which, also, there is no retirement whatever of the investment.

* *Transactions*, Am. Soc. C. E., Vol. LXXV, pp. 827–829–834–842, etc.

"The Unlimited-Life Method."—This method of making appraisals which are to serve as a basis for fixing rates, has been referred to in the writer's paper on appraisals.* In that paper the term "perpetual life" was used. It is believed to be more appropriate to use the designation here suggested.

Nearly all public service properties, taken in their entirety, may be regarded as having unlimited life. There is then no accrued depreciation to be deducted in making the appraisal. The property is kept in good condition by the repair of its parts and by replacements as these parts become useless and have to be discarded. No part of the investment is returned to the owner, but, as the plant grows old and one part after another has to be replaced, he must be allowed to recover in the earnings the cost of each article as replaced. It is on this principle that many complex, and particularly public service, properties are operated. In the early years of the life of a property made up of many parts, such as rails and ties, the replacement requirements will be small. As the property acquires age, the replacements —provided that extensions are relatively unimportant and negligible in comparison with the extent of the property under consideration— will gradually increase to nearly the amount indicated as amortization by the Straight-Line Method. The departure from this amount will be dependent on the annual extensions of the system in relation to the entire investment. There will be ultimate agreement between the replacement requirement and the straight-line amortization increment, if the plant is one that has ceased to grow. The valuation for rate-fixing purposes will always be at 100%, and the replacement requirements, until definitely ascertained by experience, will be approximated with due consideration of the age and the probable life new of the individual parts of which the property is made up.

DIAGRAMMATIC ILLUSTRATIONS.

The foregoing definitions, relating to depreciation, to replacement annuities, to the obligation to replace, and to amortization, can best be made clear by reference to the diagrams, Figs. 1 to 5.

These diagrams, being for the purposes of illustration, apply to the special case in which the article under consideration has no residual or scrap value at the time of its replacement, and in which the new

* *Transactions*, Am. Soc. C. E., Vol. LXXV, pp. 805, 815, and 875.

article which replaces it costs the same as the original article and has the same service value.

Referring to Fig. 1, illustrating the Straight-Line Method: *A B* is the original investment of capital in an article, subject to depreciation. *I J* is the portion of the original investment which has been amortized (sometimes regarded as the accrued depreciation). *K L* is the annual amortization increment. *H I* is the remaining investment. $A' B' = A B$ is the replacement cost. $B\ A'\ B'\ A''\ B''$, etc., is the investment line showing for any particular time the amount of capital still invested in the article under consideration. *M N*, in the earnings diagram of Fig. 1, represents the interest on the remaining value. $N\ O = K\ L$ represents the annual amortization.

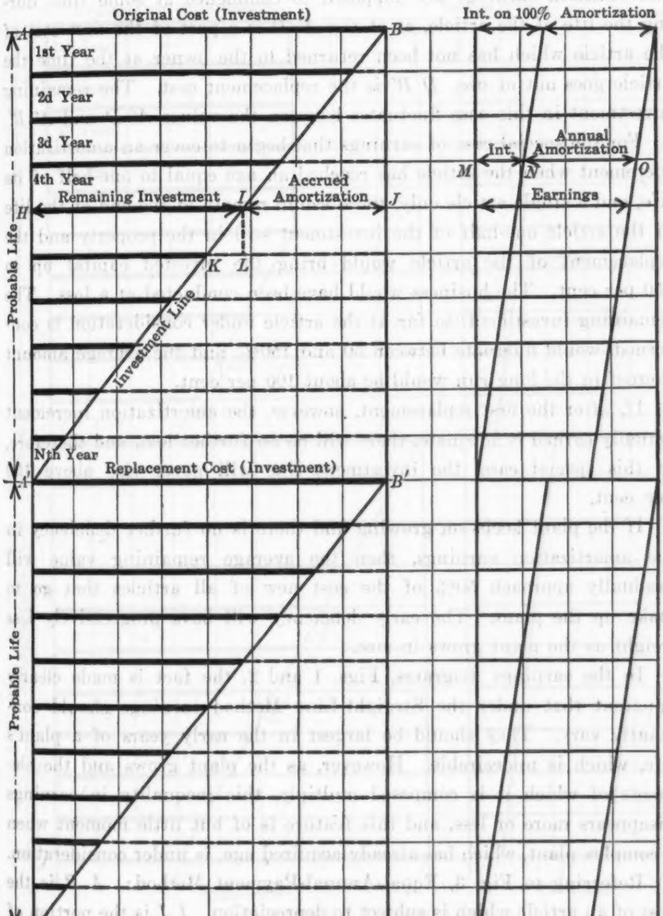
The owner is entitled to a reasonable return on the remaining investment, also to the straight-line amortization increment, $K\ L = N\ O$, besides, of course, operating expenses, including taxes, insurance, etc.

The fluctuations in the amount of capital remaining in the property are shown by the "investment line". They range from 100% for the article new to nothing at the end of its term of life. The capital returned to the owner, whether under the name of "capital amortization" or under the name of "depreciation", is his, to do with as he pleases. It is immaterial whether he places it in a special depreciation fund; whether he makes outside investments therewith; or whether, as the property develops, he reinvests it in the extensions thereof, provided, of course, that at the end of the life of the article he is prepared to reinvest the capital required to replace it. If correctly estimated and earned from the beginning, the accumulation in the fund (without interest thereon, because such interest must be regarded as outside earnings) will be just adequate to replace the worn-out article.

Under this method of amortizing capital, the earnings must be somewhat larger in the early years of the article's life than in the later ones. This is an objectionable feature.

For an infinite number of such articles, of all possible ages, the aggregate remaining investment will be 50% and the straight-line amortization increment will be just equal to the annual replacement requirement.

For a single article, in case the earning of the amortization increment does not begin until some years have elapsed, the investment line will take the shape of the broken line in Fig. 2.



STRAIGHT-LINE AMORTIZATION

FIG. 1.

Referring to Fig. 2, Incomplete Amortization Straight-Line Method: Amortization earnings are supposed to commence at some time during the life of the article, as at C. $A' D$ is a part of the first cost of the article which has not been returned to the owner at the time the article goes out of use. $D B'$ is the replacement cost. The remaining investment in this case fluctuates between the values $A' D$ and $A' B'$.

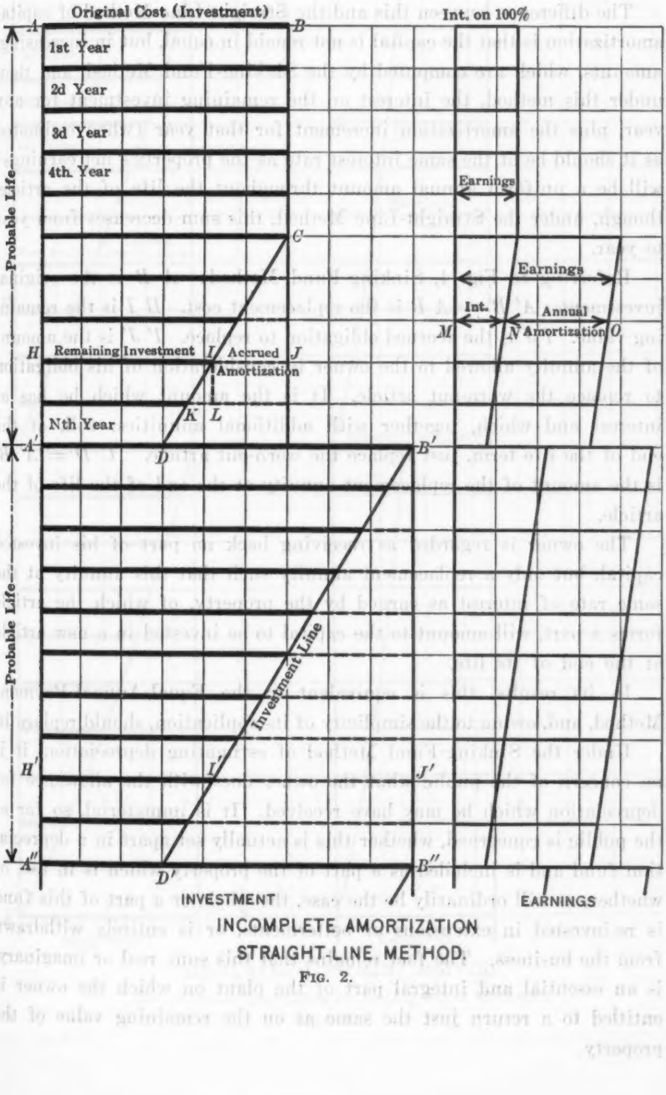
For the special case of earnings that begin to cover an amortization increment when the article has reached an age equal to one-half of its life, and a single article only, there would remain at the end of the life of the article one-half of the investment still in the property and the replacement of the article would bring the invested capital up to 150 per cent. The business would have been conducted at a loss. The remaining investment, so far as the article under consideration is concerned, would fluctuate between 50 and 150%, and the average amount thereof in the long run would be about 100 per cent.

If, after the first replacement, however, the amortization increment actually earned is adequate, there will be no further loss, and therefore, in this special case, the investment line will never rise above 150 per cent.

If the plant keeps on growing and there is no further deficiency in the amortization earnings, then the average remaining value will gradually approach 50% of the cost new of all articles that go to make up the plant. The early deficiency will have progressively less weight as the plant grows in size.

In the earnings diagrams, Figs. 1 and 2, the fact is made clearly apparent that under the Straight-Line Method earnings should constantly vary. They should be largest in the early years of a plant's life, which is undesirable. However, as the plant grows and the elements of which it is composed multiply, this inequality in earnings disappears more or less, and this feature is of but little moment when a complex plant, which has already acquired age, is under consideration.

Referring to Fig. 3, Equal-Annual-Payment Method: $A B$ is the cost of an article which is subject to depreciation. $I J$ is the portion of the original investment which has been amortized (the accrued depreciation). $H I$ is the remaining investment. $A' B' = A B$ is the replacement cost. $B A' B' A''$, etc., is the investment line representing for any particular time the amount of capital invested in the article under consideration. $M N$, in the earnings diagram of Fig. 3, represents the



INCOMPLETE AMORTIZATION

STRAIGHT-LINE METHOD

FIG. 2.

interest on the remaining investment. $N\ O$ represents the annual amortization. $M\ N + N\ O$ is a constant.

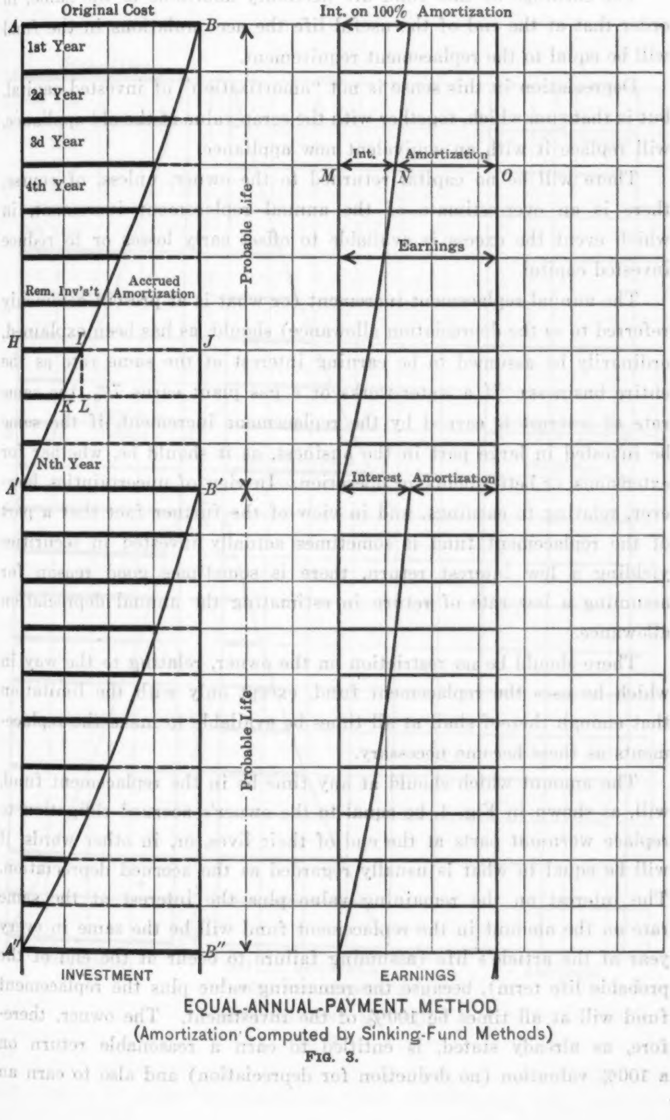
The difference between this and the Straight-Line Method of capital amortization is that the capital is not repaid in equal, but in increasing, amounts, which are computed by the Sinking-Fund Method, and that, under this method, the interest on the remaining investment for any year, plus the amortization increment for that year (when estimated as it should be at the same interest rate as the property's net earnings) will be a uniform annual amount throughout the life of the article, though, under the Straight-Line Method, this sum decreases from year to year.

Referring to Fig. 4, Sinking-Fund Method: $A\ B$ is the original investment. $A'\ B' = A\ B$ is the replacement cost. $H\ I$ is the remaining value. $I\ J$ is the accrued obligation to replace. $I'\ J'$ is the amount of the annuity allowed to the owner in consideration of his obligation to replace the worn-out article. It is the amount which he has at interest and which, together with additional annuities, will, at the end of the life term, just replace the worn-out article. $C\ D = A'\ B'$ is the amount of the replacement annuity at the end of the life of the article.

The owner is regarded as receiving back no part of his invested capital, but only a replacement annuity such that this annuity at the same rate of interest as earned by the property, of which the article forms a part, will amount to the capital to be invested in a new article at the end of its life.

In its results, this is equivalent to the Equal-Annual-Payment Method, and, owing to the simplicity of its application, should replace it.

Under the Sinking-Fund Method of estimating depreciation, it is no concern of the public what the owner does with the allowance for depreciation which he may have received. It is immaterial, so far as the public is concerned, whether this is actually set apart in a depreciation fund and is included as a part of the property which is in use, or whether, as will ordinarily be the case, the whole or a part of this fund is re-invested in extensions or betterments, or is entirely withdrawn from the business. The fact remains that this sum, real or imaginary, is an essential and integral part of the plant on which the owner is entitled to a return just the same as on the remaining value of the property.



EQUAL-ANNUAL-PAYMENT METHOD
(Amortization Computed by Sinking-Fund Methods)
FIG. 3.

The earnings of this fund are necessary additions to the same, in order that at the end of the useful life the accumulations in the fund will be equal to the replacement requirement.

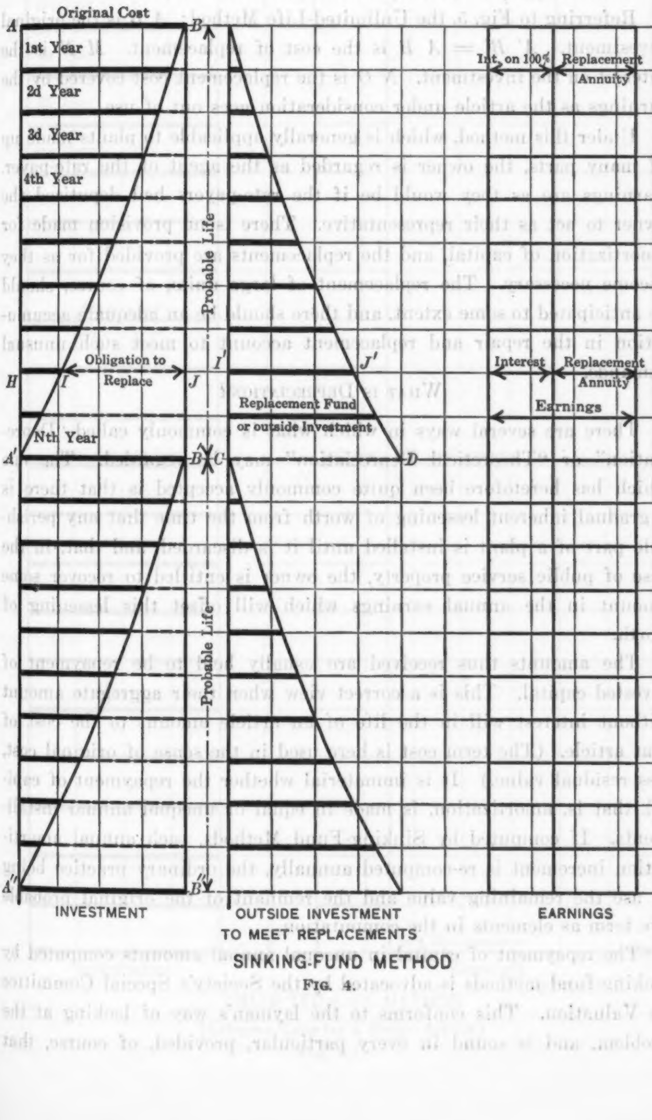
Depreciation in this sense is not "amortization" of invested capital, but is that sum which, together with the scrap value of the old appliance, will replace it with an equivalent new appliance.

There will be no capital returned to the owner, unless, of course, there is an over-estimate of the annual replacement increment, in which event the excess is available to offset early losses or to reduce invested capital.

The annual replacement increment (or what is at present commonly referred to as the depreciation allowance) should, as has been explained, ordinarily be assumed to be earning interest at the same rate as the entire business. If a water-works or a gas plant earns 7%, the same rate of interest is earned by the replacement increment, if the same be invested in large part in the business, as it should be, whether for extensions, or betterments, or operation. In view of uncertainties, however, relating to earnings, and in view of the further fact that a part of the replacement fund is sometimes actually invested in securities yielding a low interest return, there is sometimes good reason for assuming a low rate of return in estimating the annual depreciation allowance.

There should be no restriction on the owner, relating to the way in which he uses the replacement fund, except only with the limitation that enough thereof shall at all times be available to make the replacements as these become necessary.

The amount which should at any time be in the replacement fund. will, as shown in Fig. 4, be equal to the owner's accrued obligation to replace worn-out parts at the end of their lives, or, in other words, it will be equal to what is usually regarded as the accrued depreciation. The interest on the remaining value plus the interest at the same rate on the amount in the replacement fund will be the same in every year of the article's life (assuming failure to occur at the end of the probable life term), because the remaining value plus the replacement fund will at all times be 100% of the investment. The owner, therefore, as already stated, is entitled to earn a reasonable return on a 100% valuation (no deduction for depreciation) and also to earn an



SINKING-FUND METHOD

FIG. 4.

annuity which, together with interest thereon, will replace each article of which the plant is made up when it goes out of use.

Referring to Fig. 5, the Unlimited-Life Method: $A\ B$ is the original investment. $A'\ B' = A\ B$ is the cost of replacement. $M\ N$ is the interest on the investment. $N\ O$ is the replacement cost covered by the earnings as the article under consideration goes out of use.

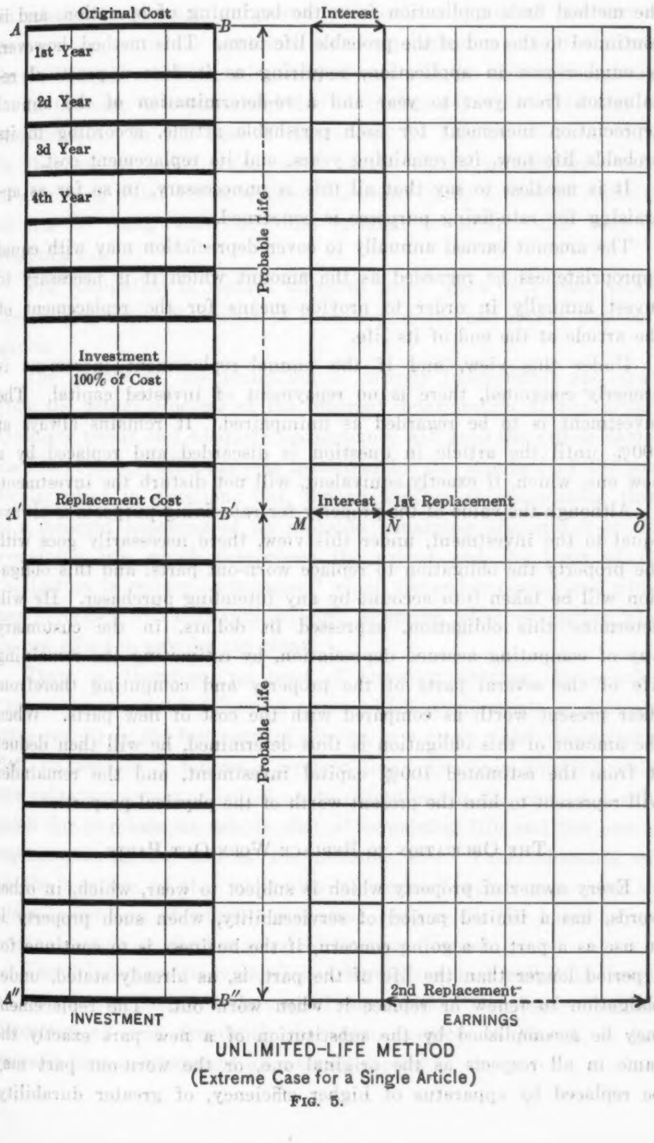
Under this method, which is generally applicable to plants made up of many parts, the owner is regarded as the agent of the rate-payer. Earnings are as they would be if the rate-payers had deputized the owner to act as their representative. There is no provision made for amortization of capital, and the replacements are provided for as they become necessary. The replacement of large units, of course, should be anticipated to some extent, and there should be an adequate accumulation in the repair and replacement account to meet such unusual demands.

What is Depreciation?

There are several ways in which what is commonly called "Depreciation" or "Theoretical Depreciation" may be regarded. The one which has heretofore been quite commonly accepted is that there is a gradual inherent lessening of worth from the time that any perishable part of a plant is installed until it is discarded, and that, in the case of public service property, the owner is entitled to recover some amount in the annual earnings which will offset this lessening of worth.

The amounts thus received are usually held to be repayment of invested capital. This is a correct view when their aggregate amount without interest will in the life of an article amount to the cost of that article. (The term cost is here used in the sense of original cost, less residual value.) It is immaterial whether the repayment of capital, that is, amortization, is made in equal or unequal annual installments. If computed by Sinking-Fund Methods, each annual amortization increment is re-computed annually, the ordinary practice being to use the remaining value and the remnant of the original probable life term as elements in the computation.

The repayment of capital in unequal annual amounts computed by sinking-fund methods is advocated by the Society's Special Committee on Valuation. This conforms to the layman's way of looking at the problem, and is sound in every particular, provided, of course, that



UNLIMITED-LIFE METHOD
(Extreme Case for a Single Article)
FIG. 5.

the method finds application from the beginning of operation, and is continued to the end of the probable life term. This method, however, is cumbersome in application, requiring as it does a practical revaluation from year to year and a re-determination of the annual depreciation increment for each perishable article, according to its probable life new, its remaining years, and its replacement cost.

It is needless to say that all this is unnecessary, in so far as appraising for rate-fixing purposes is concerned.

The amount earned annually to cover depreciation may with equal appropriateness be regarded as the amount which it is necessary to invest annually in order to provide means for the replacement of the article at the end of its life.

Under this view, and if the annual replacement increment is properly computed, there is no repayment of invested capital. The investment is to be regarded as unimpaired. It remains always at 100% until the article in question is discarded and replaced by a new one, which, if exactly equivalent, will not disturb the investment.

Although the value of the property for rate-fixing purposes is always equal to the investment, under this view, there necessarily goes with the property the obligation to replace worn-out parts, and this obligation will be taken into account by any intending purchaser. He will determine this obligation, expressed in dollars, in the customary way of computing accrued depreciation, by estimating the remaining life of the several parts of the property and computing therefrom their present worth as compared with the cost of new parts. When the amount of this obligation is thus determined, he will then deduct it from the estimated 100% capital investment, and the remainder will represent to him the present worth of the physical properties.

The Obligation to Replace Worn-Out Parts.

Every owner of property which is subject to wear, which, in other words, has a limited period of serviceability, when such property is in use as a part of a going concern, if the business is to continue for a period longer than the life of the part, is, as already stated, under obligation to renew or replace it when worn out. The replacement may be accomplished by the substitution of a new part exactly the same in all respects as the original one, or the worn-out part may be replaced by apparatus of higher efficiency, of greater durability,

of greater cost, and perhaps of greater capacity. The obligation assumed in relation to the original property is only so much of the cost of the new apparatus or appliance as is equivalent to the investment in the original article, less its residual value, at the time it is replaced.

In the case of replacement due to inadequacy, this value may be considerable. In the case of destruction by fire, by violence, or as in the case of some article such as the small, wrought-iron pipes of a growing water-works, whether discarded owing to lack of capacity, or due to being worn out, the residual or scrap value may be insignificant.

The "obligation to replace worn-out parts" goes with every public service property. No owner of such a property can escape this obligation.

The growth of this obligation is entirely independent and apart from the actual progress of the inherent deterioration of any article. Ordinary repair and adequate maintenance will keep the article during its life at 100% efficiency—as measured by the standard of the particular plant in question; but, though kept in this condition, there is a constant lessening of its probable remaining life, and it is this remaining life which determines the extent and the limit of the "obligation to replace or to renew" which the owner carries and which any purchaser will take into account, as stated, when estimating what he can afford to pay for the plant.

There is, therefore, no need of knowing whether the so-called "actual depreciation" follows a curve which drops slowly at first and more rapidly in the last years of service, or a curve which drops fast at first, or whether it follows a straight line. The only question which the valuation expert is called on to determine in the case of a valuation for purchase or sale is that of remaining life and the cost of replacement at the end of the probable life. These elements will be the measure of the "obligation to replace" which goes with the plant, regardless of whether the same remains in the hands of the original owner or whether acquired by a new owner.

The "obligation to replace" has heretofore been called by nearly all writers the "accrued theoretical depreciation" or simply "accrued depreciation" and frequently has been treated as though it had actually been a repayment of invested capital.

From an accounting standpoint, there are various ways in which theoretical depreciation may be estimated and applied in amortizing invested capital. It is unnecessary to review them. Considering physical properties alone, it is always true, on the assumption of permanency of unit costs, that the "present value" plus the "accrued obligation to replace" is just equal to the investment.

The owner is at all times entitled to a return at a fair rate of interest, both on the present value and on the obligation which he has assumed to keep the plant at 100% valuation. Consequently, when rates are to be fixed, there is no need of attempting to separate the investment into its two parts, "present or depreciated value" and "accrued depreciation", of which the latter might better be called "the outside investment necessary to provide for replacements." There is nothing to be written off under this method of treatment from the capital account for "accrued depreciation", the accrued depreciation being nothing else than an estimate of the amount of an "investment necessary to provide for replacements" and this may be regarded as the "accrued obligation to replace", expressed in dollars.

THE PRESENT OR REMAINING VALUE AS AN ELEMENT IN REGULATING RATES.

When a property is to be purchased, a prudent purchaser will make inquiry relating to the condition of the various parts, in order to ascertain the amount of deferred maintenance and the probable additional term that the perishable parts of the property will serve. He will also ascertain the probable cost of replacing each part, with due allowance for residual values. He will thus obtain a measure of the obligation, which he assumes, to replace worn-out parts and to keep the plant at 100% efficiency. He will, as here indicated, treat the "obligation to replace worn-out parts" just the same as he will treat deferred maintenance. Both of these items he will deduct from "cost to reproduce new." The remainder is what he can afford to pay for the physical plant.

There is, of course, nothing wrong in using the value of physical properties as determined in this way (practically investment-less-depreciation) as a basis for the regulation of rates, provided that methods are correctly and continuously applied throughout the entire life or probable life term of each perishable article; but there will be from year to year a change in the remaining life of the several

parts of a plant, with a consequent change of remaining value and a change in the annual depreciation increment. Computation by any method requiring an annual re-determination of depreciated value, becomes involved and complex, and by its volume burdensome.

It is to be hoped that these methods, and particularly the method recommended in the Report of the Society's Special Committee on Valuation, already referred to, will not be generally accepted, and that the simpler and exactly equivalent Sinking-Fund Method of valuing always at 100% of the investment, as set forth in the writer's valuation paper,* will be substituted for the Committee method, or that, better yet, the Unlimited-Life Method will be adopted whenever conditions favor its application.

Valuation at 100% of the Investment as an Element in Regulating Rates.

The application of these simple principles, which involve the assumption that no portion of the earnings is used to retire capital, but that a suitable part of the earnings will go into a replacement fund or into an investment for replacement purposes, can perhaps best be understood on the basis of an example:

An electric generator with a 20-year life may be used for illustration. Assume, in the absence of any accepted method of appraisal, that remaining value is determined by deducting accrued depreciation estimated by sinking-fund methods.

Suppose the generator to be a part of a light and power system, and suppose, further, that it has reached the last year of useful life and will have no scrap value. A purchaser will value the generator, if he estimates interest on a 6% per annum basis, at about 8.23% of its original value, and this is all that he will pay for it. He takes upon himself, when he buys the light and power plant, an obligation to replace the generator with a new one at the end of another year. He must then renew the investment represented by this article at 100 per cent. The obligation which he voluntarily takes upon himself at the time of his purchase to replace the generator in a year is 91.77% of the cost of a new generator. At the end of the life of the generator, he will have received in his earnings the last increment of the original investment in the generator, or 8.23% of its cost, and he will meet

* *Transactions*, Am. Soc. C. E., Vol. LXXV, pp. 770 *et seq.*

his obligation to continue in business by acquiring a new generator and thereby renewing the full original investment of 100% in this particular appliance.

It makes no difference whether there is only one generator, or whether there are twenty in use. The principle is always the same. Moreover, in such a simple case as that of twenty generators of all possible ages, each with a useful life of exactly 20 years, the accrued obligation to replace these generators when worn out (6% per annum interest being again used for purposes of illustration) will be 41%, as shown by the writer's formula.*

In other words, the remaining value of the twenty generators (cost less depreciation, as ordinarily noted) will be about 59 per cent. No purchaser would include the twenty generators in his valuation of the property at more than 59% of their aggregate cost; but he would, nevertheless, and with reason, expect to be allowed to earn interest on 100% of their cost new, claiming rightfully that he is entitled to the same rate of income return on accumulated annual replacement increments as he is entitled to earn on the rest of his invested capital. He will justify this claim by pointing out that the earnings on any accumulated replacement fund are not available for any other use than the replacement of worn-out property; that such earnings do not, therefore, represent income; and that it is for this reason he is entitled to have the aggregate annual replacement allowances, together with accumulated interest, treated as interest-bearing capital.

In this connection attention may be called to the laws of California,† which prescribe that the Railroad Commission (having the duties of a Public Service Commission)

"may from time to time ascertain and determine and by order fix the proper and adequate rates of depreciation of the several classes of property of each public service utility. Each public service utility shall conform its depreciation accounts to the rates, so ascertained, determined and fixed, and shall set aside the moneys so provided for, out of earnings and carry the same in a depreciation fund, and expend such fund only for such purposes and under such rules and regulations, both as to the original expenditures and subsequent replacement, as the Commission may prescribe. The income upon investments of moneys in such fund shall likewise be carried in such fund."

* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 780.
† Statutes of 1911, 1st Extra Session, Ch. 14, Sec. 49.

It is here apparently recognized that the depreciation fund is to be used for the specific purpose of replacing worn-out parts, and that, in regulating rates, consideration need be given only to the amount of capital reasonably and properly invested, without any deduction for depreciation.

The earnings, in other words, to be adequate, must include operating expenses of every character, a proper allowance for present and prospective replacements (so-called depreciation), and a reasonable income computed at the proper interest rate on the invested capital.

It has been amply demonstrated in the writer's paper on appraisal, that this method of valuing for rate-fixing purposes at 100% of the invested capital is identical in its results with Method No. 3 of that paper, which is the method of computing depreciation recommended in the report of the Special Committee on Valuation, and which, while apparently conforming to the Court decisions, is yet, like others, not strictly correct unless it has been applied from the beginning. When these methods have not found application from the beginning of operation, the transition is easy from either of them to the assumption that a public service property has unlimited life and that all replacements are to be treated the same as ordinary repairs and maintenance.

According to the system of valuing properties for rate-fixing purposes at 100% of the reasonable and proper investment, it is not necessary to ascertain the accrued depreciation or the equivalent amount that should be in a replacement fund, or that should be regarded as invested to provide for renewals. It is necessary, however, in estimating earnings, to determine how much thereof should be allowed to the owner to compensate him for assuming the obligation to replace. This, if estimated for a new plant under the Sinking-Fund Method, is the annual increment determined by sinking-fund compound-interest methods, which, in the years of the probable useful life at the same rate of interest as is earned (net) by the entire property, will amount to the cost of replacing the parts of the property when worn out.

The allowance for the obligation to replace, when operating on the assumption of unlimited life of the entire property, is the actual annual replacement requirement, which need not be closely estimated, because the surplus of any one year will be available for use in subsequent years.

If the owner of a plant of numerous parts and all possible ages is required to place all sums earned as "depreciation" or earned as a "replacement annuity", in a special fund, the theoretical balance on hand in such a fund will be, as here noted:

Under the Straight-Line Method of estimating amortization for plants of full growth, the accumulation will be 50% of the investment in perishable property; this accumulation is independent of the interest rate, and the earnings by this fund are of no concern to the rate-payer.

Under the method of estimating the "accrued obligation to replace" by the Sinking-Fund Method for old plants, with interest at 6%, and the hypothesis that actual life will conform to probable life,* the theoretical accrued obligation to replace will be:

For articles having			5-year life, about				38%.
"	"	"	10	"	"	"	40%.
"	"	"	20	"	"	"	38%.
"	"	"	30	"	"	"	34%.
"	"	"	40	"	"	"	31%.
"	"	"	50	"	"	"	28%.
"	"	"	60	"	"	"	25%.

These percentages apply strictly, of course, only to cases in which the age of the plant is greater than the life of its parts. They indicate the extent, however, to which an accumulating replacement or depreciation fund may exceed the demands on it. In some measure, approximating this unexpended surplus, such a fund is available for outside investments. Interest on the same, when it represents the obligation to replace, must be earned by the owner for the benefit of the plant; consequently, it makes no difference to what purpose he applies this surplus, provided only that ample assurance remains that replacements will actually be made when necessary.

When the capital is being amortized, the accumulations in the amortization fund are the absolute property of the owner. They are not subject to control by the rate-payer, yet, nevertheless, here, too, proper assurance may be expected that replacements will be duly made, and it would not be unreasonable to expect an adequate fund to be maintained to meet emergencies.

* *Transactions*, Am. Soc. C. E., Vol. LXXV, formula on p. 784.

In order to show beyond all question that the method of valuing at 100% in fixing rates under continuous application from the beginning is correct and proper, an electric generator with a life of 20 years, which has served 15 years, may be taken as a basis for an illustration. The usual assumption is made, for the purpose of this illustration, that there is no change in the cost of this article during its life and that it will go out of service when exactly 20 years old. Interest in this illustration is taken at 6% per annum:

Equal-Annual-Payment Method.—(Always correct if applied from the day the article went into use.)

Original investment........................		$100.00
Life (new).............20 years		
Time in service.........15 years		
Remaining life........... 5 years		
Accrued depreciation (= amortization to date).		63.27
Remaining value...........................		36.73
Interest on remaining value............	$2.20	
Annual depreciation or annual amortization increment for 16th year........	6.52	
Required net earnings............	$8.72	

In this case the depreciation in the 16th year is that amount which invested annually at 6% will retire the remaining value $36.73 in the remaining 5 years of life.

Sinking-Fund Method.—(Method No. 1, as explained in the writer's paper on Valuation.*)

Permanent investment.......................		$100.00
Life (new)............20 years		
Time in service..........15 years		
Remaining life.......... 5 years		
Interest on the investment.............	$6.00	
Annual depreciation or annual replacement increment for any year........	2.72	
Required net earnings............	$8.72	

* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 770.

Although it can thus be shown that for each year the earnings should be $8.72 on the $100 of original capital investment, a new calculation is necessary for each perishable article and for each year under the Equal-Annual-Payment Method, and for the rational Sinking-Fund Method a single simple calculation suffices for each group of articles of the same probable life term.

Effect of Inadequate Earnings.

Depreciation or replacement annuities are not ordinarily fully covered by the earnings in the early years of the life of public service properties. The cases in which earnings are adequate from the beginning to yield a proper interest return, together with the replacement annuity, are relatively few. Consequently, particularly if methods of appraisal other than the Unlimited-Life Method are insisted upon, there may be a period in the life of the public service concern during which a part of the so-called accrued depreciation will remain unrecovered. Temporarily at least, there will then appear to be a business loss; but a loss of the character which, in the course of time, should be amortized if the business enterprise is a legitimate one.

The United States Supreme Court, in Knoxville *vs.* Knoxville Water Company, says:

"If, however, a company fails to perform this plain duty and to exact sufficient returns to keep the investment unimpaired, whether this is the result of the unwarranted dividends upon over-issues of securities, or of omission to exact proper prices for the output, the fault is its own."

In expressing this opinion, the Court has overlooked the fact that such earnings as it assumes to have been possible can rarely be realized at the beginning of a new business, and that there will be some years in the life of almost every public service concern for which it would be unjust to make so sweeping an assumption.

Although it would be improper to consider early losses, including deficient or unrecovered depreciation, in every case, as the exact amount to be amortized by future earnings, there is no doubt that due consideration should be given to such losses in order that, from the sum of experience, the world over, a proper basis may be established for making the necessary allowance therefor. This allowance, of course, can be in the nature of an interest-bearing addition to the invested capital.

It does not, strictly, represent value until a return thereon in the earnings is assured.

It is to be expected, therefore, that in the case of many public service properties the owner will have had no opportunity to make investments the earnings of which will meet his replacement requirements. When there has been such a deficiency of earnings, the ordinary method of computing the annual replacement increment by sinking-fund, compound-interest methods, based on the original probable life or on the reduced value computed by any method and the remaining life, would be improper and inadequate. It is under such circumstances that recourse must be had to special methods of computing the depreciation or the annual replacement increment.

Unlimited-Life Method.

In such cases as those just referred to it may be necessary to provide for a replacement at the full cost of various parts in the remaining years of life, or of burdening the property with an allowance for the accrued deficiency of earnings, which is then to be amortized in a reasonable number of years.

When an appraisal is to be made, for rate-fixing purposes, of a plant which has continuously been operated with inadequate earnings, there is no escape from the conclusion that the investment has remained undiminished. It would be unreasonable not to concede to the owner his right to a return on the investment and also to allow him to earn enough in addition to meet replacements as they become necessary. Considerations of this character led the writer to adopt the Unlimited-Life Method as not only fair to both owner and rate-payer, but as the one method which has the peculiar advantage of requiring lowest earnings in the early years of operation when the rate-payers are fewest in number.

This method has the advantage over all the other methods that it is applicable regardless of past conditions. All the other methods, the Straight-Line Method of estimating depreciation, the Equal-Annual-Payment Method, the Sinking-Fund Method, are only then correct and equitable methods of making appraisals and of determining annual depreciation, or amortization increments, when they are properly applied from the beginning of operation. Their use assumes continuous appli-

cation throughout the entire probable or actual life of each part of a plant.

When the Court says that, if an owner fails "to exact sufficient returns to keep the investment unimpaired, whether this is the result of the unwarranted dividends * * * or of omission to exact proper prices for the output, the fault is his own", this fact is overlooked. It is time that the Courts revise such decisions, as they have already caused much trouble and no little juggling on the part of experts to secure justice for their clients.

There can be no fairer nor better way of determining the earnings of a public service property than that under which the owner is regarded as the agent of the public. He is then to be allowed interest on the investment which he makes as such agent, and he is also to be allowed whatever may be necessary for the up-keep of the property, maintenance, repairs, and replacements.

There should be no amortization of capital, no more than there would be if the rate-payers themselves made the investment without the intervention of an agent.

If this view is adopted, every public service property may be considered to have unlimited life. The property in its entirety is not subject to depreciation. Its appraisal at all times for rate-fixing purposes is at the full amount of the investment.

The earnings must be adequate to provide interest on the full amount of capital invested and also to meet the replacement requirements. The annual replacement requirement will be negligible in the early years of a plant's life, or in the early years of any single article; but this replacement requirement will grow as the plant acquires age. It can be tentatively estimated. All sums collected for replacements should be separately accounted for. Ordinarily, the replacement of larger units will be anticipated, and their cost will be distributed over a number of years, and, in the long run, particularly when a complex plant worth millions is under consideration, there should be no difficulty in estimating a fair average annual replacement requirement.

What Part of Earnings May Be Written Off Capital?

Under the impression that any sum collected under the designation "depreciation" or "amortization" is a repayment of a part of the investment, such sums are frequently written off from the in-

vested capital or from the "cost of reproduction new", which is an approximation of the invested capital.

This is correct:

1st.—When, from the beginning of the installation, the Straight-Line Method of estimating "amortization" or "depreciation" is applied and the amortization allowance has actually been earned.

2d.—When the actual amount of the annual decrease in remaining value has been computed by the Equal-Annual-Payment Method and earned from the beginning.

3d.—When any other definite scale of capital repayment is adopted such that in the life of the part the several payments "without interest thereon" amount to 100 per cent.

Value from Various Standpoints.

Under the Straight-Line Method of computing depreciation or of amortizing capital, the value for rate-fixing purposes, "cost less depreciation", is the same as value to a purchaser. This is self-evident.

Under the Equal-Annual-Payment Method, the appraisal for rate-fixing purposes is the cost, less accrued depreciation; and here, too, this value is the same as that which would be determined by a purchaser.

Under the Sinking-Fund Method, the appraisal for rate-fixing purposes is at 100% of the investment. It is assumed, under this method, that the replacement fund is intact and a part of the property. Thus considered, the property (including the replacement fund) has a value to a purchaser of 100 per cent. If he does not pay 100% and does not get possession of the replacement fund, then he must put it up himself, because he takes with the property, as has been explained, the obligation to replace, which, expressed in dollars, is the amount which should be in the replacement fund.

Under the Unlimited-Life Method there has been no return of capital. The appraisal is at 100% of the investment. The value of the property to a purchaser is 100%, because he takes the place of the original owner and is entitled to receive interest on the full amount of the investment. The earnings yield him interest on this amount, and he needs no replacement fund, because the replacements will be met out of earnings from year to year as they have to be made.

It thus appears that value for rate-fixing purposes and value to a purchaser can be reconciled, regardless of the method adopted for computing adequate earnings.

Appraisal Must be Made with Due Regard to Past History of the Property and to Probable Future Conditions.

A change from one method of computing allowable earnings to another is not always possible without injustice to either the rate-payer or the owner. If the Equal-Annual-Payment Method has been acceptable for a time and it is then determined to pass to the Straight-Line Method, there will be an immediate reduction of the interest-bearing portion of the investment. The cost-less-depreciation will be smaller under the latter than under the former, and the owner will be compelled to sacrifice a part of his investment without compensation.

If the Sinking-Fund Method has been used in computing allowable earnings and there is a change made to the Straight-Line Method, this will release the accumulation in the replacement fund, which is thereupon to be regarded as returned capital, but the replacement fund, whether it exists as a fund or has been otherwise used by the owner, will fall short of the accrued depreciation, as the same would have been estimated by the Straight-Line Method, and, consequently, the owner will again be unfairly deprived of a part of his investment.

If the Sinking-Fund Method has been used from the beginning of operation and a change is made to the Unlimited-Life Method, there may be an accumulation in the replacement fund, or there has been distributed to the owner from this fund a larger sum than is necessary to aid in equalizing the annual actual replacement requirements, and to this extent the owner would benefit by the change.

If operations have been conducted from the beginning on the basis of earning a proper interest return on the investment, plus the amount necessary from year to year to renew worn-out parts, and there is then to be an appraisal and computation of rates by any method other than the Unlimited-Life Method, this would be an injustice to the owner, because he has not in the past had the benefit of the larger earnings which he should have had under these other methods of computation.

All this is referred to, to show that the correctness of any method of appraisal for rate-fixing purposes is not predicated on the method alone, but on the continuous application of the adopted method from the beginning, and on the assurance that the same method will be continued in the future. The decisions of the Courts cannot change

this fact, and, where they have disregarded it, they will undoubtedly be modified in the course of time.

Comparison of Actual with Probable Life.

Attention has been called* to the fact that the expectancy of articles which remain in use, instead of probable life new, should be taken into account whenever an article has acquired some age and has survived the possible accidents of the early period of its life. In other words, it is proper to pay attention to the condition of the plant and to the condition of the articles of which it is made up when remaining value is to be estimated.

All estimates of annual depreciation and of accrued depreciation are based on premises which cannot be determined with accuracy. The probable life of any article when new and the life expectancy of any article which has been in use for some time cannot be determined with any great degree of precision. Consequently, all estimates of depreciation are only approximations.

There is so much uncertainty in such estimates, under the ordinary conditions under which public utilities are operated, that a wide range in the method of making the estimates has been the result.

When, therefore, the correctness of methods is under discussion, this fact should not be lost sight of. Nevertheless, the academic discussion which is being indulged in by the Engineering Profession is justified, because it will lead to an ultimate standardization of methods and finally to the general adoption of the most convenient and generally best method of making the valuation which is to serve as the basis for fixing rates fair to both the rate-payer and the owner.

Recognizing this limitation, and all the uncertainties that result from imperfect knowledge relating to the actual and to the probable life of the elements of any public service plant, and to the difficulty of determining the expectancy of those elements which have already been in use for some time, it is nevertheless important that the whole question be fully considered from the theoretic standpoint, in order that a framework may be constructed into which the best data furnished by experience can be fitted.

* *Transactions*, Am. Soc. C. E., Vol. LXXV, pp. 837-838.

The non-agreement of the actual life of individual items with the probable life and the extent to which this should be taken into account in estimating present worth and in estimating replacement requirements have, in line with this thought, been studied on various assumptions with interesting results. These will be briefly referred to, the tables that were prepared being withheld except as to final results until more definite information is available relating to the probable rate of failures from year to year with advancing age in any group of articles.

When any large number of articles which have the same probable life, as, for example, 10 years, are considered, there will be as many aggregate service years represented by the failures to reach the probable life term of 10 years as there will be service years represented by those articles which outlast the 10-year term. It may also be accepted as a certainty that there will be a greater number of articles per year to go out of use in the years just preceding and just following the term limit than at any other time.

This is in conformity with the law of probabilities.

On this hypothesis, with the assumption of failures according to the law of probabilities, but all within a period equal to twice the probable life, and on another hypothesis according to which a gradual uniform increase of failures up to the tenth year and thereafter a gradual uniform decrease of failures to the twentieth year—or to twice the probable life term, it was found possible to predict expectancy and also to compute the annual replacement requirement.

Unfortunately, there are no records available to justify the assumption that all failures may be assumed to occur within twice the probable life term. Nevertheless, the distribution of failures to the double term, with a greatest rate of failures at the end of the probable term, is certain to be far nearer the truth than the usual assumption of general agreement between probable and actual life. It may be proper, therefore, to refer to the results of the studies.

Under the assumption of failures according to the law of probabilities, it was found that an article having a probable life of 10 years when new should have the expectancies shown in Table 1.

On the assumption that the rate of increase in the annual number of failures is uniform from the beginning of service to the end of the probable life term, and that there is a similar uniform decrease

in the annual number of failures after the end of the probable life term, Table 2 has been prepared, and shows the expectancy for any surviving article that is still in good condition.

TABLE 1.—EXPECTANCY, ACCORDING TO THE LAW OF PROBABILITY, FOR AN ARTICLE WITH A 10-YEAR PROBABLE LIFE; ON THE ASSUMPTION THAT ALL ARTICLES FAIL WITHIN A PERIOD PRACTICALLY TWICE THE PROBABLE LIFE TERM.

No. of years.	Expectancy, in years.	No. of years.	Expectancy, in years.
1	10.00	11	2.71
2	9.00	12	2.42
3	8.05	13	2.18
4	7.11	14	1.97
5	6.22	15	1.80
6	5.40	16	1.64
7	4.78	17	1.50
8	4.04	18	1.25
9	3.52	19	1.00
10	3.08		

Although, under this hypothesis of failures, there may be considerable departure from the actual number of failures in any year, there can be no question that this hypothesis is a much nearer approach to the truth than the assumption that remaining value and accrued depreciation can be estimated from tables based on probable life without regard to the condition of an article under consideration.

TABLE 2.—EXPECTANCY ESTIMATED ON THE ASSUMPTION THAT, OF A LARGE NUMBER OF ARTICLES HAVING A 10-YEAR PROBABLE LIFE, THE MAXIMUM RATE OF FAILURES IS 10% IN THE TENTH YEAR, WITH A GRADUAL INCREASE TO THIS AMOUNT FROM THE FIRST YEAR AND A GRADUAL DECREASE FROM THIS AMOUNT TO THE TWENTIETH YEAR.

No. of year.	Expectancy in years.	No. of year.	Expectancy, in years.
1	10.00	11	3.67
2	9.09	12	3.33
3	8.27	13	3.00
4	7.46	14	2.67
5	6.77	15	2.33
6	6.12	16	2.00
7	5.51	17	1.67
8	4.95	18	1.33
9	4.44	19	1.00
10	4.00		

The basis for the results in Table 2 is shown diagrammatically in Fig. 6. The expectancy is found by dividing the remaining service years at any time by the corresponding number of surviving units. The reverse curve, marked "Remaining Units", clearly indicates the hypothesis of failures on which the table is based. It is to be noted that, under this hypothesis, there is no serious departure from the results that were obtained by assuming that the law of probabilities would apply.

The replacement requirements, if failures occur substantially as assumed, would increase, for $100 of original investment, from $1 in the first year to about $10 in the ninth year, fluctuating thereafter between $9 and nearly $12 per year, and gradually settling down to $10 per year. For an annual investment of $100 per year (that is, for a growing plant) the replacement requirements would gradually increase from $1 per year in the first year to $463 in the fiftieth year, or from $1 per $100 of investment in the first year to $6.01 in the tenth year; to $8.16 in the twentieth, and to $9.27 in the fiftieth year.

In practical application, in other words, the annual replacement requirement in the case of a plant of full growth, all parts of which have a probable life of *n* years after the plant is *n* years older than any of these parts, will be about one-*n*th of their replacement cost.

In a plant which continues to grow, the annual replacement requirement will gradually approach, but can never quite reach, one-*n*th of the total replacement cost.

Although it may be granted that in the long run the failures of individual articles in any class will follow some definite law (perhaps a law similar to that which has been cited as more probable than failure always at the end of the probable life term), the fact remains that in no particular case, no matter how large a plant may be, will there be absolute conformity with any such law. Consequently, figures determined on the basis of any hypothesis of failures can be used to prepare smoothed-out curves, and from such curves tables for general use can be prepared. It is enough to know for the present that, ordinarily, it may be assumed that the replacement requirement of a large number of articles with a probable life of *n* years should increase progressively year by year to about one-*n*th of their cost in the *n*th year.

EXPECTANCY DIAGRAM; TEN YEAR PROBABLE LIFE; 1000 UNITS.

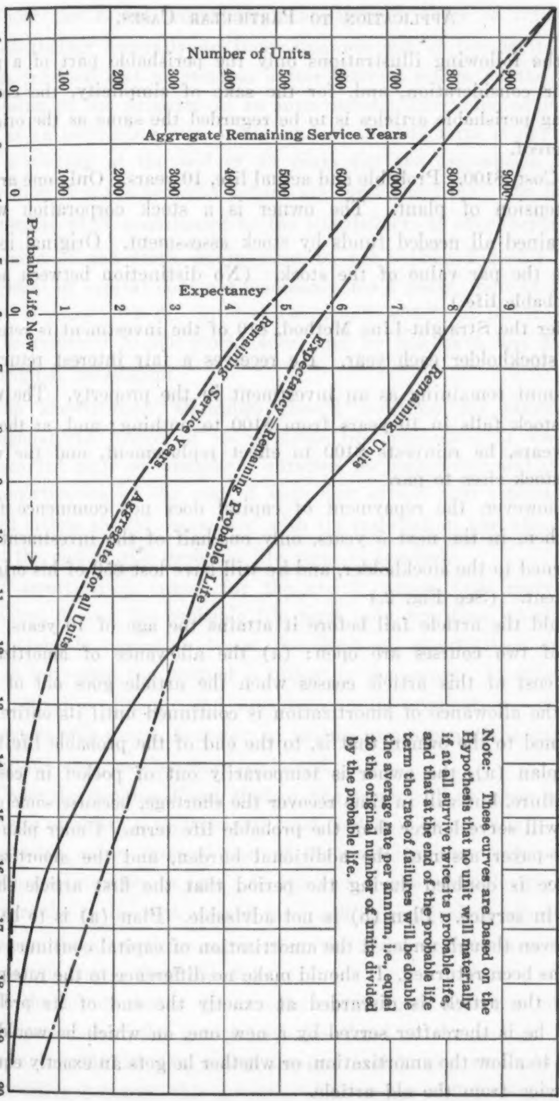


FIG. 6.

Application to Particular Cases.

In the following illustrations only the perishable part of a plant is under consideration, and, for the sake of simplicity, the cost of replacing perishable articles is to be regarded the same as the original cost thereof.

1.—Cost, $100. Probable and actual life, 10 years. Only one article. No extension of plant. The owner is a stock corporation which has obtained all needed funds by stock assessment. Original investment is the par value of the stock. (No distinction between actual and probable life.)

Under the Straight-Line Method, $10 of the investment is returned to the stockholder each year. He receives a fair interest return on the amount remaining as an investment in the property. The value of his stock falls in 10 years from $100 to nothing; and, at the end of 10 years, he reinvests $100 to effect replacement, and the value of his stock rises to par.

If, however, the repayment of capital does not commence for 5 years, then, in the next 5 years, only one-half of the investment will be returned to the stockholder, and he will have lost $50 of his original investment. (See Fig. 2.)

Should the article fail before it attains the age of 10 years, then either of two courses are open: (a) the allowance of amortization on the cost of this article ceases when the article goes out of use; or (b) the allowance of amortization is continued until its entire cost is returned to the owner, that is, to the end of the probable life term. Under plan (a), the owner is temporarily out of pocket in case of early failure, but will in time recover the shortage, because some other article will serve longer than the probable life term. Under plan (b), the rate-payer assumes the additional burden, and the amortization allowance is doubled during the period that the first article should still be in service. Plan (b) is not advisable. Plan (a) is to be preferred, even though under it the amortization of capital continues after 100% has been returned. It should make no difference to the rate-payer whether the article is discarded at exactly the end of its probable life and he is thereafter served by a new one, on which he would not hesitate to allow the amortization, or whether he gets an exactly equivalent service from the old article.

Under the Equal-Annual-Payment Method, there is returned to the stockholder annually an increasing amount of the capital which he has invested and he receives a fair interest return on the amount remaining in the property as an investment. The value of his stock falls at an increasing rate as the end of the life term is approached, and is nothing at the end of 10 years (on the assumption that no distinction is made between actual and probable life). On the reinvestment of $100, contributed by the stockholders to effect replacement, the value of the stock again rises to par. (See Fig. 3.) If the repayment of capital does not commence for 5 years, then in the next or last 5 years on a 6% interest basis, only $57.23 of the investment will be returned to the stockholder, and he will have lost $42.77 of his original investment.

The amortization allowance should continue for every article throughout the period of its probable life, regardless of whether it is discarded early or whether it survives the full term; and then, too, the amortization of the cost of each new article that replaces a discarded article should commence with its installation and should cease when the end of the probable life term is reached.

If amortization ceases for the article which fails early and is continued during the probable life term only for those which outlast their probable age—this being the limit of amortization tables as now in use—then the method is unjust to the owner. In other words, this method is difficult and uncertain in its application. Some improvement will result, however, in taking into account the expectancy at varying ages, as elsewhere explained.

Under the Sinking-Fund Method, the owner will be given an opportunity to establish a replacement fund, and if it be actually established and treated as an integral part of the property, the stock value will remain continuously at par. At the end of the probable life term of 10 years the entire replacement fund will be required to effect replacement, on the supposition again that the failure of the article occurs just at this time. (See Fig. 4.)

If the accumulation in a replacement fund does not commence for 5 years, then the amount in this fund at the end of 10 years will be only $57.23 (on a 6% interest basis), and the value of the stock will not exceed this amount. The owner will have lost $42.77.

It need hardly be stated that here, too, expectancy should be taken into account, when a determination of remaining value is to be made.

Under the Unlimited-Life Method, the replacement will be assured at the end of 10 years and the value of the stock at all times will be at par. (See Fig. 5.)

2.—Cost, $100. Probable and actual life, 10 years. Only one article. No additions are made to the plant. The owner is a stock corporation which has constructed the plant with money obtained by a bond issue.

Under the Straight-Line and Equal-Annual-Payment Methods, bonds are retired as amortization allowance is earned. The stock has only such value as results from an interest allowance in excess of the interest on the bonds, and no value if these interest rates are the same, and if, as assumed in this illustration, there is no surplus of earnings above interest on remaining investment, plus the amortization allowance. In case that the amortization is not earned for a term of years, the ownership of stock may become a liability instead of an asset.

Under the Sinking-Fund Method, the fund which is accumulating for replacements may be used for the retirement of bonds or otherwise. The stock will again have only such value as results from interest earned in excess of interest on bonds. In the case of earnings for a term of years, which are inadequate to meet the replacement annuity, the stock may be a liability instead of an asset.

Under the Unlimited-Life Method, bonds are refunded as they become due. The amount of outstanding bonds remains unchanged. The stock has only such value as results from the excess of interest earned when compared with the interest on the bonds.

It is obvious from these illustrations that, when works are constructed under bond issues, the earnings should exceed the interest on the bonds, in order to insure some compensation for management and the hazards of the business.

3.—Cost, $100. A single article with a probable life of 10 years. Interest, 6 per cent. With due regard to condition or to the departure of actual life from probable life.

Under the Straight-Line Method, the invested capital is reduced $10 each year. At the end of the 10-year term the article is found to be still in good condition; its expectancy is then probably from 3 to 4

years. The owner has already received back his full investment. There is no remaining investment, although the remaining value of this unit is from \$30 to \$40. By some hard and fast rule, the \$10 per year payment should continue until the article is discarded, or by some other equally inflexible rule, as under illustrations (1) and (2), the \$10 per year should continue until ten payments have been made, and should cease to be paid after the tenth payment. (See Table 3.)

TABLE 3.—Remaining Value and Required Earnings.

Straight-Line and Equal-Annual-Payment Methods. Cost, \$100. Single Article. Probable Life, 10 Years. Interest, 6 per cent. Net Earnings, 6 per cent. Applicable to a Surviving Unit.

(Expectancy on the Hypothesis Explained in the Text.)

Year.	Expectancy, in years.	Remaining Value.*		Required Earnings.	
		Straight-line method.	Equal-annual-payment method.	Straight-line method.	Equal-annual-payment method.
1	10.00	100.00	100.00	16.00	13.59
2	9.09	90.90	93.09	15.46	13.59
3	8.27	82.70	86.54	14.98	13.59
4	7.46	74.60	79.77	14.50	13.59
5	6.77	67.70	73.77	14.08	13.59
6	6.12	61.20	67.89	13.66	13.59
7	5.51	55.10	62.11	13.30	13.59
8	4.95	49.50	56.73	12.94	13.59
9	4.44	44.40	51.55	12.64	13.59
10	4.00	40.00	47.08	12.40	13.59
11	3.67	36.70	43.50	12.22	13.59
12	3.33	33.30	39.91	11.98	13.59
13	3.00	30.00	36.32	11.80	13.59
14	2.67	26.70	32.51	11.62	13.59
15	2.33	23.30	28.75	11.38	13.59
16	2.00	20.00	24.91	11.20	13.59
17	1.67	16.70	20.87	11.20	13.59
18	1.33	13.30	16.85	10.78	13.59
19	1.00	10.00	12.82	10.60	13.59
20					

* At the beginning of the tenth year, the expectancy is 4 years, consequently, the remaining value of the article under the Straight-Line Method at \$10 per year is \$40.

In a 10-year life sinking fund table at 6% the remaining value for 4 years of remaining life is \$47.08. The annuity, which will amount to \$47.08 in 4 years, is \$10.77. Interest on \$47.08 at 6% is \$2.82. The required net earnings, therefore, are \$10.77 + 2.82 = \$13.59.

Under the Equal-Annual-Payment Method, the remaining value at any time may be estimated from 10-year life tables, using the expectancy as an element in the calculation. If the probable life of the article new is used throughout, this should be done only under an inflexible rule, and all amortization should cease when the article

has reached the end of the probable life term, as under illustrations (1) and (2). If, however, the condition of the article is taken into account and its expectancy is determined, the remaining value will be about as shown in Table 3.

Under the Sinking-Fund Method, with a replacement fund made due regard to the expectancy of the individual articles, the required annual earnings will be uniform in amount and the same as under the Equal-Annual-Payment Method.

Under the Unlimited-Life Method, the investment again remains unimpaired, and whenever the article fails, whether early or late, it is replaced with a new one, the replacement allowance to be earned being the amount already noted, in order that funds may be available when the failure occurs.

4.—Cost, \$100. Numerous articles. Probable life 10 years, some articles failing early, others serving after the end of the probable life term. The plant has full growth, and there are no further additions or extensions. All computations on a 6% interest basis.

Under the Straight-Line Method, the annual allowance for amortization will be \$10. At the end of the 10 years, there will be some of the original articles still in service, possibly 45 to 50 out of each 100. The others, making up the original group, having an aggregate cost of \$100, have failed and were replaced with new articles and some of these have failed and were replaced. The owner has received amortization in the sum of \$100, and he has reinvested in replacements somewhat more than \$60. The remaining investment, therefore, is about or a little more than \$60. After the first 10 years have passed, the annual replacement requirement will be just about equal to the amortization allowance, and the remaining investment will continue at some amount somewhat in excess of \$60. The required earnings on a 6% basis begin with \$16 per year, gradually dropping to between \$13 and \$14.

Under the Equal-Annual-Payment Method, the appraiser will find himself in considerable difficulty, until the time is reached when the number of annual failures is fairly uniform. The extent of the amortization will then be about \$30 to \$40, and the remaining investment will be about \$60 to \$70. The replacement requirement in this case, also, will be just about equal to the annual amortization allowance, and the remaining investment will continue at about \$60

to $70. (The required earnings will be uniformly and continuously $13.59 per year.)

Under the Sinking-Fund Method, with a replacement fund made up of equal annual replacement allowances computed by the Sinking-Fund Compound-Interest Method, there will be a gradual growth of this fund in 10 years, until the number of annual failures is fairly uniform, when there will be about $30 to $40 in the fund. Thereafter, the interest on this fund, together with the annual replacement allowance, will be about $10, which is the amount necessary to meet the annual replacement requirement. The required earnings are, from the beginning and continuously, $13.59 per year. The replacement fund will gradually grow until it contains about $30 to $40. This fund will be available for outside investments.

Under the Unlimited-Life Method, the $100 of original investment would remain unimpaired, and there would have to be provided a growing annual amount for actual replacements, beginning with about $1.00 the first year; $2.01, the second; $3.04, the third; and so on, to about $10 in the tenth year and thereafter.

5.—The case remains to be considered of a plant which has not attained full growth, and to which additions are made from year to year.

From a tabular comparison it appears that, when computed with regard to the expectancy of surviving articles, the application of the Unlimited-Life Method has the particular advantage of requiring lowest earnings in the early years, and the Straight-Line Method has the disadvantage of requiring the highest earnings in the early years; the Equal-Annual-Payment Method holds an intermediate position. The required earnings, computed on the hypothesis of failures as heretofore stated, would be $7.00, $16.00, and $13.59, respectively, in the first year; $120.12, $143.38, and $135.90, respectively, in the tenth year; and $283.18, $281.36, and $271.81 respectively, in the twentieth year.

Comparison of Methods.

When a municipality constructs improvements under a bond issue, suitable provision is made for the replacement of any of the worn-out parts of these improvements at the time that such parts go out of use. This is in strict conformity with the proceedings under the Unlimited-Life Method; but, in the case of the municipality, the

bond issue is also to be taken care of. The cost of the improvements is, in other words, to be distributed fairly to those who will benefit by them. This is usually done by fixing the term of the bonds so that the cost will be distributed over a sufficiently long period of time. The determination of this time period need not be in any definite relation to the life of the parts of the improvement. The improvement itself will usually be one that may be regarded as having unlimited life, such as parks, playgrounds, streets, and the like. When the term of the bonds of longest life has been fixed, on the basis of the probable life of the main elements of the improvements or otherwise, then a determination must be reached as to the best and most equitable rate of amortization.

This amortization may take place at a uniform rate per year, bearing heavily on present-day property owners—Straight-Line Method.

It may take place at an increasing rate per year:

(a) According to the scheme outlined under the Equal-Annual-Payment Method; or

(b) According to any arbitrary scheme that will approximate the compound-interest Sinking-Fund Method of estimating the annual amortization increment.

Or, it may be deferred for a time and then take place according to either of these methods.

In the case of a public service property constructed by a municipality, the amortization of capital usually begins at or soon after the acquisition of the property; and in the case of a utility constructed by a private owner, the amortization of capital should begin under the Straight-Line and Equal-Annual-Payment Methods at the beginning of operation, and, under the Sinking-Fund or the Unlimited-Life Methods, as herein fully explained, not at all during the continuance of private ownership.

In order to make clear how unsatisfactory it is to have to estimate "depreciation", or, strictly speaking, "the replacement requirement", according to any method except that of unlimited life, the diagrams on Fig. 7 have been prepared.

The curves here shown afford a comparison of replacement requirements determined according to some reasonable hypothesis relating to the expectancy of surviving articles, with the amortization allowance

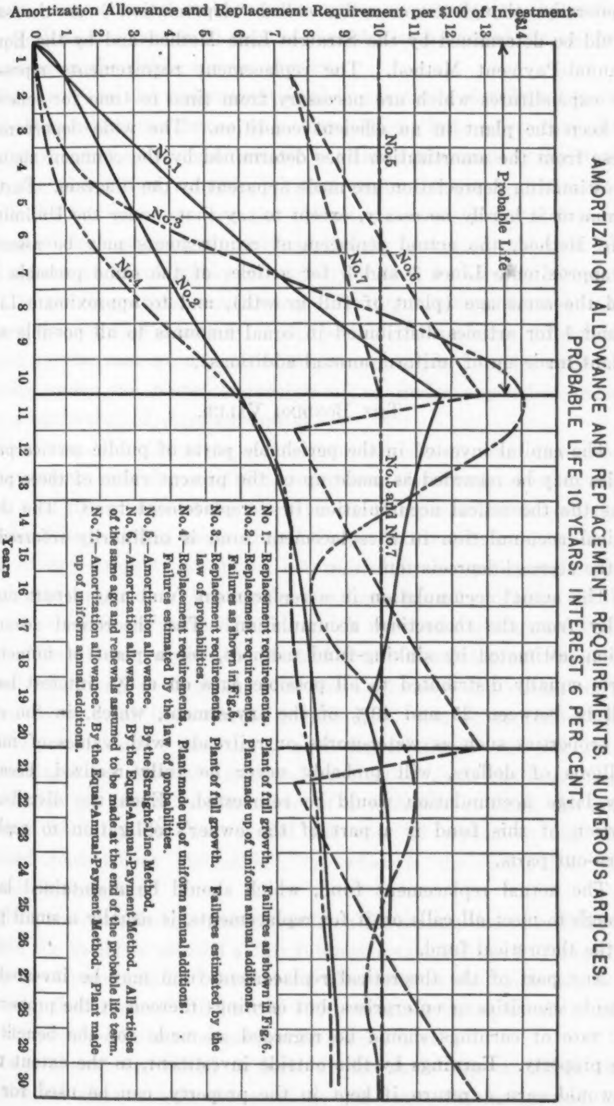

AMORTIZATION ALLOWANCE AND REPLACEMENT REQUIREMENTS. NUMEROUS ARTICLES.
PROBABLE LIFE 10 YEARS. INTEREST 6 PER CENT
Amortization Allowance and Replacement Requirement per $100 of Investment.
$14
Probable Life
Years
No. 1
No. 2
No. 3
No. 4
No. 5
No. 6
No. 7
Nos. 5 and No. 7
No 1.—Replacement requirements. Plant of full growth. Failures as shown in Fig. 6.
No. 2.—Replacement requirements. Plant made up of uniform annual additions. Failures as shown in Fig. 6.
No. 3.—Replacement requirements. Plant of full growth. Failures estimated by the law of probabilities.
No. 4.—Replacement requirements. Plant made up of uniform annual additions. Failures estimated by the law of probabilities.
No. 5.—Amortization allowance. By the Straight-Line Method.
No. 6.—Amortization allowance. By the Equal-Annual-Payment Method. All articles of the same age and renewals assumed to be made at the end of the probable life term.
No. 7.—Amortization allowance. By the Equal-Annual-Payment Method. Plant made up of uniform annual additions.

Fig. 7.

(generally, though erroneously, called "depreciation") as the same would be determined by the Straight-Line Method and by the Equal-Annual-Payment Method. The replacement requirements represent the expenditures which are necessary from time to time for renewals to keep the plant in an efficient condition. The wide departure of these from the amortization lines determined by the common methods of estimating depreciation are made apparent by the diagram. Further comment is hardly necessary, except to say that, under the Unlimited-Life Method, the actual replacement requirements may be assumed to approximate Lines 1 and 3 for articles of the same probable life and the same age (plant of full growth), and to approximate Lines 2 and 4 for articles distributed in equal amounts to all possible ages (plant made up of uniform annual additions).

The Bonding Value.

The capital invested in the perishable parts of public service properties may be regarded as made up of the present value of these parts plus the theoretical accumulation in a replacement fund. The theoretical accumulation in a replacement fund is ordinarily referred to as the accrued depreciation.

The actual accumulation in a replacement fund may depart materially from the theoretical accumulation. The theoretical accumulation, estimated by sinking-fund methods, for a plant of numerous parts equally distributed to all possible ages on a 6% interest basis, will be between 25 and 40% of the investment, which, in the case of properties such as water-works or railroads, with values of many millions of dollars, will probably never be fully realized, because any large accumulation would be reinvested. Even the distributed portion of this fund is a part of the owner's obligation to replace worn-out parts.

The actual replacement fund, which should be maintained large enough to meet all calls on it for replacements, is usually a small part of the theoretical fund.

Any part of the theoretical replacement fund may be invested in outside securities or enterprises, but earnings thereon at the property's net rate of earnings should be regarded as made for the benefit of the property. Earnings by this outside investment, to the extent that it would earn a return if kept in the property, can be used for no

other purpose than to make replacements; consequently, the entire theoretical replacement fund may be regarded as a part of the owner's investment in the public service property.

The surplus in the replacement fund is sometimes distributed to the stockholders when the owner is a corporation; but, when thus distributed, it is not a dividend, because it reduces the value of the stock. When the distribution is made, whether under the name of dividend, or not, the fact remains that the obligation of the individual share of stock to replace worn-out parts is thereby increased and that any such increase of the obligation must carry with it a lessening of worth.

It is not correct to consider an annuity which, together with interest compounded annually, will, within the probable life term, amount to the cost of a perishable part of a plant, when collected annually under the name "depreciation" or under any other name, to be a repayment of invested capital.

When the depreciation or replacement annuity is thus computed, the investment remains at 100% regardless of what is done by the owner with the accumulating depreciation or replacement allowances.

A bond issue should never exceed the present worth, as the present worth of any property would be determined by a purchaser; consequently, the valuation of a property as a basis for a bond issue should take into account the method under which required earnings are computed. When the Straight-Line Method or the Equal-Annual-Payment Method has found application from the beginning, the valuation as a basis for a bond issue should include the "cost to reproduce the physical properties new less the accrued depreciation". So, too, when the Sinking-Fund Method has found application from the beginning, but without the actual establishment of a replacement fund, the outstanding bonds should not exceed the cost to reproduce less the "accrued replacement obligation", or, what is the same, less the accrued depreciation.

When, however, the appraisal is made on the assumption of unlimited life and the owner is assured a fair interest return on the full amount of his investment besides replacement allowance as the same becomes necessary, the property will have a 100% bonding value, as all extensive, complex, public service properties should have.

Bonds should be retired at least at the same rate at which the amortization allowance in the earnings will retire the investment, or,

in the case of the Sinking-Fund Method, at the same rate at which the obligation to replace grows.

In a property all parts of which have a limited life, when earnings are estimated on any other assumption than that of unlimited life for the entire property, the outstanding bonds may approach the 100% limit when the property is new, but should gradually fall to about 60% of the cost to reproduce new when the plant has reached an age greater than the probable life of any of its parts.* If bonds are issued in excess of this amount, and the property is turned over to the bondholder to cancel the obligation, he would find the value of the property inadequate to meet the indebtedness with which it is burdened; the bonds could not maintain a par value.

There are, of course, cases where bond issues are required to replace worn-out parts. In such cases the funds provided by the bond issue restore the particular parts which are replaced to a 100% value, and this full value, and not the remaining value of the old parts, at the time of commencing proceedings for the issue, is the proper basis on which to determine the permissible limit of the issue.

The use of the "replacement" or "depreciation" fund for retiring bonds is a proper use. It makes no difference whether any large surplus in this fund is allowed to accumulate, and is invested in interest-bearing securities, whether it is used in extending the plant, or whether it is used to discharge a debt on which the owner is paying interest, provided only that an adequate portion of the fund is readily convertible when required to pay for replacements.

Special Notes.

It is evident from what has here been presented that any attempt to estimate the amount of amortization allowance which should be made, to keep pace with the theoretic depreciation, is, at its best, only a crude approximation; that, if the plan to be followed is one which is to deal with depreciated value and not with the full investment, care must be exercised, not alone in adopting a method of making capital repayments, but also in determining how much of past earnings can be construed as having been applied, first to amortize early losses, cost of establishing business, and, second, to amortize capital, and in determining therefrom the remaining investment.

* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 784.

To avoid these uncertainties and to establish a better relation of the earnings of the utility to the ability of the rate-payer to pay, recourse should be had to the method of valuation designated as the Unlimited-Life Method, whenever applicable, thereby eliminating all necessity for close approximations of depreciation. This method has easily recognized advantages, both in the case of old, well-established properties, and in the case of properties hereafter to be established.

In fixing rates, if amortization of capital is determined on, cognizance must be taken of the adopted plan of amortization.

Either (A), the amortization continues during the actual life of the article whether less or greater than its probable life new;

Or (B), the amortization ceases at the end of the probable life term, regardless of whether the article covered by the amortization has failed early or whether it continues in use long after the end of its original probable life term.

These are correct methods of amortizing capital, whether in uniform annual amounts, in increasing annual amounts, or according to any other plan.

When franchises have a definite time limit, at the end of which the plant is to be disposed of to the public, there may be some special provision for the transfer. If the transfer is to be made without any payment at the end of the franchise term, or at any elected time under a franchise with indeterminate life, then the invested capital should be returned during the time that the plant is in private ownership. If, on the other hand, the transfer is made, as it should be, on the basis of an investment by the public in the property at the time it is taken over, then the remaining investment should be determined according to the appraisal method that has been in vogue during the life of the property, and this remaining investment, which may be 100% under the Sinking-Fund and Unlimited-Life Methods, or less than this under other methods, will be the amount which the original owner should receive.

The past history of a public service property should always be taken into account when an appraisal is made which is to serve as a basis for fixing rates. It is unjust to assume that a part of the investment has been amortized unless the records or other satisfactory evidence shows this to have been the fact.

When rate-regulating bodies, such as Public Service Commissions, lay down rules requiring appraisals to be made of depreciated value (particularly when amortization, under the guise of depreciation, is subtracted from cost), grave injustice may result therefrom to the owners of established public service properties, and it is notably in such cases that experts exercise their ingenuity to bolster up values.

There are difficulties all along the line when appraisals are made by any other than that which has here been called the Unlimited-Life Method.

Depreciation is not amortization. It has nothing to do with the repayment of invested capital, and should always be considered apart from amortization.

It is difficult, usually unnecessary, and almost impossible to compute accrued or current depreciation correctly.

It is generally undesirable to amortize the investment in a privately owned public utility.

It is undesirable and yet sometimes necessary to include in appraisals value for intangible elements, such as early losses, or cost of establishing the business, as is frequently done, particularly when depreciated values are used as a basis for determining allowable earnings.

It is undesirable to permit appraisals for rate-making that will cause fluctuations in earnings, except only such as show a gradual increase of aggregate earnings, keeping pace, as time goes on, with the increasing number of rate-payers.

The service rendered should be at all times up to an established standard, and the price of the service should be fairly stable, with a downward rather than an upward tendency.

The bonding value should be as stable as circumstances will permit. This is secured by adopting the Unlimited-Life Method of making appraisals for rate-fixing purposes whenever it is applicable.

The Equal-Annual-Payment Method should be regarded as impracticable, except under inflexible rules making probable life the prime consideration, or requiring that due regard be had to expectancy. In the former case, the amortization will begin for each new investment as made and will continue for the assumed probable life term and no longer. Then, under such a rule, there will be times when amortization will be allowed on some article already out of service

at the same time that the cost of the article which replaces it is being amortized, and there will be other times when the cost of an article is completely amortized, and no amortization installment is to be provided for. Where single articles, or plants made up of very few articles, are under consideration, the undesirability of such a rule is apparent. Under the alternative rule, the method becomes difficult and inconvenient to apply. The Sinking-Fund Method is always to be preferred.

There are cases, as, for example, in a steamboat business with only one steamboat in service, where the advantages of the Sinking-Fund Method, as herein explained, over the other methods of appraisal, are readily apparent.

The Straight-Line Method is generally undesirable, because, if applied from the beginning, it entails early high aggregate earnings which must be collected from only a few rate-payers, and because its application is only correct when it has been in continuous use from the beginning.

The Unlimited-Life Method makes all considerations of depreciation, except for the purpose of tentatively approximating replacement requirements, unnecessary. It reduces the services required of valuation experts and accountants. It should be insisted on by the owner whenever it is applicable, and it should be elected by the rate-payer because it is fair and will make the required earnings least in amount when there are fewest rate-payers, that is, in the early years of operation.

The decisions of the Courts in defining fair value and in declaring for depreciated value are not to be taken as conclusive. They will be modified when these matters are better understood.

DISCUSSION

Mr. Mortimer.

JAMES D. MORTIMER,* M. AM. SOC. C. E. (by letter).—Mr. Grunsky is quite correct in asserting that the inclusion of allowances for depreciation in operating expenses, when testing for reasonable rates, is essentially different from the amortization or return of invested capital to the proprietors of a public utility. The obligation to make replacements of worn out, inadequate, or obsolete plant runs concurrent with the operation of the utility. These facts were pointed out in the writer's discussion on the Report of the Special Committee on Valuation of Public Utility Properties, to which reference is now made.† The indeterminate nature of the problem of future life of a particular unit of equipment was also pointed out, and the similarity of the estimates of depreciation allowances to those of the life insurance actuary was indicated.

1.—After a unit of equipment has become part of an operating plant, the essential difference in value between that of an old and of a new unit lies in the probability of the former requiring earlier replacement or abandonment than the latter. The old unit may serve its purpose just as well as would a new unit of the same or of an improved type. The difference in relative value must accordingly be measured by some factor that has to do with the time of its future replacement. This factor can be most conveniently measured as that reserve which will insure the replacement of the unit at the end of its probable life time. This thought has led to the development of the expression, "Replacement Insurance", to cover the concept ordinarily described as "Depreciation."

2.—If a utility has accrued an adequate reserve for replacement insurance and the same is properly recorded on the books of the corporation, the property value or capital for purposes of rate-making should contain no deduction for depreciation in whatever manner it may be measured. If, however, the corporation fails to record this liability on account of replacement insurance and disburses the amount allowed in earnings therefor, in the form of interest and dividends on its capital obligations, there apparently results a withdrawal of capital from the utility, and the fact that "Depreciation" should not be deducted is not so clear.

3.—Regulating commissions and Courts rarely have to deal with cases involving public utilities in which there has been a complete reservation for replacement insurance. The problem usually met covers the case where the reservation has been only partial, or, more frequently, where the reservation has not yet been begun. The cases contemplated in Mr. Grunsky's hypothesis, accordingly, are the exception rather than the rule.

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* New York City.

† *Proceedings*, Am. Soc. C. E., for April, 1914, p. 1174.

Mr. Mortimer.

4.—Although every public utility corporation has the implied obligation to make replacements of worn out, inadequate, or obsolete units of plant, as they become necessary or advisable, it is not always readily evident that such corporations possess the financial ability to fulfill such obligations. If the assets of such a corporation should be mortgaged for the purpose of securing an issue of bonds, many cases arise where the obligation to make replacements cannot readily be enforced against the bondholders. Capital stock of such corporation is usually issued "full paid and non-assessable", and the obligation to make replacements cannot in such cases be enforced by assessment of the holders of capital stock. In view of these legal obstacles, it is not surprising that regulating commissions and Courts have been inclined to deduct the contingent obligation for replacement from the estimated cost new of the property, when considering valuations for rate-making.

5.—When the property constituting a utility is transferred from one stock corporation to another, or to a municipality, the transfer presumably carries with it the implied obligation of future replacements of physical plant. The purchaser, in assuming this obligation, will either require that it be shown by assets other than physical property or deduct the amount of such obligation or liability from the purchase price that would obtain were there no such obligation. This would seem to indicate a possible difference in value for the two purposes of rate-making and sale.

6.—The rate cases which have been before the United States Supreme Court have invariably been presented on the theory of confiscation, as this is the only basis on which such cases may be brought before the Federal Court. The idea of confiscation contemplates the acquisition of the whole or a part of the property. Thus, rate cases before the Federal Courts automatically become purchase cases, and, in view of this fact, the dicta of the Supreme Court are essentially applicable to valuation for purposes of sale. In such cases, however, there appear to be important elements of value, all of which are referred to in the Smythe *vs.* Ames Case, but which have obtained only slight recognition in the actual monetary measurement of value.

7.—As value for purchase and sale may depend on earnings, and earnings in turn may depend partly on rates of charge for service supplied by the utility, it is not unnatural that regulating commissions should have endeavored to appraise the capital investment devoted to public use by some method which would be independent of current earnings, that is, independent of the rates of charge for service. Although the definitions of principles of valuation are of great importance, from the standpoint of law, economics, and social equity, the making of reasonable rates of charge for service depends on many factors, of which property valuation is only one. The final test of

Mr. Mortimer. reasonable regulation cannot be formulated with mathematical exactness, but will be judged by the attitude of the proprietors of the utility affected toward the investment of additional capital. The reasonableness of regulation may be approximately ascertained without an attempt at valuation, as is shown by the present-day situation of the steam railway carriers.

8.—It seems readily evident that the regulation of public utilities may be justified in law without introducing the doctrine of "principal and agent", to which the author makes incidental reference, and which has achieved some popularity with economists of a certain class. This theory is wholly unnecessary to a proper definition of all the principles of regulation, and unfortunately possesses a slant in a direction toward which there is no good reason for regulation to tend.

Mr. Mayer. JOSEPH MAYER,* M. AM. SOC. C. E. (by letter).—The author gives two definitions of depreciation. He says, under "Depreciation", that it is to be measured "by a comparison between the probable life, the expectancy, and the cost of replacement at the expiration of the term of the article's actual serviceability".

Under "Remaining Value" or "Present Value", he says:

"This is the present worth of any article, dependent on, and to be computed from, the three elements: (*a*) the probable useful life of the article new; (*b*) its probable remaining life; and (*c*) the cost of replacement, which, when prices are not subject to change, is the original cost less the residual value of the article when it goes out of use."

When prices are subject to change, as is nearly always the case, the author defines "Replacement Cost" by: "The cost of effecting a change from the worn-out part to a new part of equal service value." He evidently means of equal service value with the worn-out part when it was new.

The author does not state how the computation of the remaining value is to be made, but one would naturally infer that, when there is no residual value, the remaining value is the cost of replacement multiplied by the probable remaining life and divided by the probable useful life of the article new. Or:

$$\text{Remaining value} = c\,\frac{b}{a}.$$

The author evidently believes that interest has nothing to do with the case, but, later, he mentions obsolescence and inadequacy as influencing the probable remaining life. The definition contains two serious errors. The article has a certain annual service value, a, during an estimated number, n, of years. If i is the rate of interest,

* Montreal, Que., Canada.

if the residual value of the article is zero, if a and the value of the new article are constant, if $f = \frac{100}{100 + i}$, and if the annual service values are received at the end of each year, then the total service value, S_n, of the article, which is the sum of the present values of all its annual service values, is Mr. Mayer.

$$S_n = af + af^2 + \cdot \cdot \cdot \cdot \cdot + af^n = af\,\frac{1 - f^n}{1 - f}.$$

If the probable remaining life of the old article is r years, then the total service value of the old article is:

$$S_r - af\,\frac{1 - f^r}{1 - f},$$

and the ratio of the two values is:

$$\frac{S_r}{S_n} = \frac{1 - f^r}{1 - f^n}.$$

Therefore, if the given assumptions are fulfilled, it is not necessary to determine the constant annual service value of the article in order to find the ratio of the values of the old and the new articles. According to the author's definition,

$$\frac{S_r}{S_n} = \frac{r}{n}.$$

He would make the remaining value of an old article, which has a probable remaining life of 20 years and a probable life when new of 60 years, of one-third as much value as the value of the new article; but, with 6% interest, as $f = \frac{100}{106}$, the writer obtains

$$\frac{S\,20}{S\,60} = \frac{1 - \left(\frac{100}{106}\right)^{20}}{1 - \left(\frac{100}{106}\right)^{60}} = 0.710.$$

The real value of the old article, with the assumptions made, is therefore 71%, and not one-third of the value of the new one. The depreciation is 29%, and not 66⅔%, as given by the author's definition.

The assumptions agree with those made by the author. They agree, however, but seldom, if ever, with the facts. Most articles lose in efficiency of service with age, and the cost of their maintenance generally increases toward the end of their useful life, until the net revenue, a, derived from the use of the article, becomes zero, when, provided it has no residual value, it will be discarded. If it has a residual value, it will be discarded sooner and used for another purpose. Therefore, the annual service value, a, is generally not a constant, and may

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suddenly change when a much cheaper equally serviceable or useful article for the attainment of the same end comes into the market. Obsolescence, therefore, affects not only the probable remaining life, but also the annual service value, of an article. Though the appearance of a cheaper article of equal usefulness does not modify at all the material usefulness of the old article, it does reduce the value of this usefulness, as its value depends on its cost.

This applies also to an obsolescent plant as a whole; its remaining value as a plant does not exceed the cost of a modern plant of equal usefulness. The land of the plant, together with its parts, however, may be more valuable than the plant; it will then probably be dismantled.

The value, S_n, of an article is measured by its market price. The equation,

$$S_n = a\, f \frac{1 - f^n}{1 - f},$$

$$\text{gives } a = \frac{S_n}{f} \frac{1 - f}{1 - f^n},$$

$$\text{where } f = \frac{100}{100 + i},$$

i is here the annual profit, and a is the cost per annum of the article. The use of an article is only justified if its annual usefulness exceeds the annual cost, a; in this case its value in the plant is equal to its cost.

For an article, S_n is the price, which can be easily ascertained, and therefore a can be found when n and i are known. For a business enterprise, there is generally no known market value, S_n, but its net revenue, a, can be ascertained. If a is variable, then

$$S_n = a_1 f + a_2 f^2 + - + (a_n + R) f^n$$

where a_1, a_2, . . . a_n, are the succeeding annual revenues, and R is the residual value of the plant when it is abandoned.

Besides depreciation, the author discusses appraisal. Appraisal means the finding of the value of a thing. What is wanted is the fair value, or the value correctly ascertained. The Courts say that the fair value is the present value, and that investment or original cost and additions of cost to date, cost of reproduction new less depreciation, past, present, and prospective revenue, market value, and possibly other factors, must be considered in ascertaining this value. They maintain that there is only one fair present value, and not different values for different purposes.

The value of a thing, as defined by economists, and, on the whole, accepted by the highest Courts, is the quantity of other things it

will buy, when the buyer and seller are good judges of value and in a position to act in accordance with their judgments of values. Mr. Mayer.

From the standpoint of the investor, a public utility is a tool to secure a net revenue. The investors or possible buyers of such a utility judge its future net revenue from its recent and present net revenues and the causes of changes in the future.

If the utility is subject to regulation, its net revenue is influenced to a large extent by this regulation, and its future value depends on the nature of the regulation. The investors are mostly convinced that the Courts will protect present values of past investments, and will not permit their confiscation by a change in the regulation.

The present fair value of a public utility, therefore, is given by a correct estimate of the present value of the future net revenues which would be obtained with the present regulation.

Any legal regulation must pay the past investors a fair return on the present value, or it must give them the present value.

The author says:

"'Fair Value', then, is necessarily based on proper and reasonable investment which may be ascertained from cost records, and, when cost records are not available, is usually estimated by the 'cost-of-reproduction' method."

Values change with conditions and laws. The values of products supplied by many competitors approach cost of production, which includes cost of labor and of use of capital, rent, taxes, and other incidental expenses, and very variable profits. The profits of each producer, together with the other costs which he incurs, must result in such prices of the articles produced that they can be sold in competition with similar articles offered by other producers. The average profits in all industries, but not the individual profits of different competitors in them, tend, with free competition, toward equality. With free competition, very different profits are obtained by the creators of individual industrial plants, and any estimate of value based either on actual costs, or cost of reproduction, assumes uniform profits for the creation of plants or business enterprises, whether they are planned, built, and managed by the highest industrial talents or by mediocrities. It is, therefore, necessarily erroneous, even if it is made with the utmost care and without any other error.

Recently the bondholders of a nearly new railroad, 90 miles long, in Western New York, proposed to dismantle it because it could not be operated without loss. This railroad is worth less than one-fifth of its actual cost or its cost of reproduction. Such extreme cases are rare, but very large differences between values and either actual cost or cost of reproduction with uniform profits are the rule and not the exception. Natural monopolies existed for a long time without regu-

Mr. Mayer. lation. High prices and excessive profits often resulted therefrom. This produced a rush of capitalists to acquire such monopolies, and many enterprises were started long before they could be made profitable. The investors expected to be repaid for early losses by later large profits. If they succeeded, the public had to pay for the unnecessary losses of premature enterprises. Competition for the acquisition of the monopolies did exist, but excessive prices, nevertheless, often resulted.

This led to the regulation of the prices of monopolized products. The general endeavor was to establish such prices as would give the same return on invested capital in monopolized as in competitive industries. Before regulation existed, each enterprise obtained what profit it could get, and the profits on the actually invested capital differed very widely, according to the intelligence and skill of the promoters and managers of the enterprises. The values of the enterprises which arose under such conditions are neither equal to the investment nor to the cost of reproduction less depreciation. They are governed by estimates of the present values of the future net revenues, and they can be ascertained only by such estimates, and not by either of the two methods advocated by the author. Neither of these methods is sustained by the higher Courts, and no sane buyer or seller would follow either of them. It is often arbitrarily assumed that, because the actual future revenue is dependent on the future regulation, revenue cannot be used as the basis of the valuation for the purpose of rate-making. This is bad logic. As the present values, protected by the Courts, are those which would prevail with the present regulation, they are independent of the future regulation as long as this protection lasts. As the properties, in the future, unless assisted by the public, must earn a fair return on their present values, the future regulation depends on the present values.

The fair return may be much smaller than the present one, if the present owners are given safe securities of the same market value as their properties. Keeping promises is the foundation of all honest business, and this is what the Courts insist on. The method of valuation advocated by the writer is in substantial agreement with the decisions of the highest Courts. Both the methods defended by the author, if adopted, would revolutionize all existing values of public utility companies. That which he prefers would give for old properties results farthest from the actual values. This method is not at all defined by him, as he would fix arbitrarily what part, if any, of the development costs would be included and what part of deficient or excessive past income would be considered.

Mr. Grunsky endeavors to describe and improve a hopelessly confused industrial system which has arisen from empirical attempts to

establish, by means of regulation, just prices of monopolized products. He advocates the principle that the private owners of public service properties should receive remuneration in proportion to the investment. Mr. Mayer.

The author defines Investment as "the aggregate of the reasonable and proper expenditures which have been made to render the property in question efficient for the purpose for which it is intended." From the standpoint of the public the purpose of a public utility is to give a product or service, of the desired quantity and quality, at the lowest possible price. The reasonableness of any expenditure depends on its efficiency to secure its ends, or on whether its usefulness justifies its cost. This depends mainly on ascertaining correctly the difficulty and importance of each part of the work, fixing an adequate compensation in proportion thereto, securing the best men obtainable for it, in judging the merits or the usefulness for their purpose, of the various materials needed, and selecting the cheapest when all effects are considered.

There are only two ways of securing high efficiency of expenditures, or, what amounts to the same thing, a considerable amount of reasonableness: One is to put a capable man at the head of an enterprise, with full power to select the proper means for prescribed ends, and to make him responsible for results; the other is to make the compensation of the independent owners proportional to the efficiency they secure. These independent owners may then be trusted, with little supervision, to select fit men, but they cannot be trusted to do this if their compensation is in proportion to their investment. In this latter case, the risk is with the public; full control must go with the risk. This means public management of promotion, construction, and operation, or, in effect, public ownership. If there is private management of promotion, construction, and operation, no outsider can judge correctly the reasonableness of expenditures. He cannot possibly secure a high degree of reasonableness; that can only be obtained by the actual manager in charge. The manager, therefore, must be appointed and discharged by the representative of the parties most interested in the efficiency. This party is, with the author's method, the public, and it must appoint the managers.

The author's method of controlling and rewarding private owners of public utilities is impracticable and unreasonable. Therefore, it is impossible to make it consistent and definite. Though he endeavors to secure for the owners a fixed return on the investment, he is obliged to define the investment so that it is not the amount of money honestly spent by the investors, but something else, which cannot be ascertained correctly with private management. He is also unable to define when inadequate earnings to secure the desired rate of interest on the investment, either because the regulation was imperfect or because even

Mr. Mayer. without regulation the property could not earn the interest allowed, shall entitle the owner to recover past losses by future earnings.

If an earthquake or a flood destroys part of the investment, the author leaves it in doubt whether or not the destroyed investment is entitled to future earnings.

The decision of all these questions is left in the hands of authorities elected by the consumer, subject to control by the Courts.

The contract between the owner and the consuming public is so extremely indefinite that nobody can know what is a fair construction of it. Under such circumstances, justice becomes impossible, and innumerable lawsuits can only be prevented by making them so expensive that both parties will submit to a very large amount of injustice rather than go to law. Capital, therefore, can be secured only by promising an excessive rate of interest. The investor in such enterprises is practically in the position of a lender of money to a man with very doubtful credit; he is not sure whether he will obtain the promised interest or will ever get back the principal. Capital can be obtained at a moderate rate, if the public takes all the risks and adopts full public ownership wherever a definite contract, leaving the risks with the private owner and giving him only a fair return, is impracticable.

For this purpose the value of such utilities as are best publicly owned and operated must be determined by estimating the present value of their future earnings with the present method of control, provided this control cannot be shown to be illegal and unfair. New bonds, with a moderate rate of interest, can then be issued to pay with their proceeds to the present owners the values of their properties. The properties should then be managed by general managers with full control, responsible to boards of directors, who prescribe the ends to be attained, watch the efficiency of the manager, and are responsible to the public. As money can be obtained on bonds under normal conditions at from 4 to 5%, the change to public ownership would secure, with proper valuations and good public management, a large saving to the consumers.

In all his calculations the author neglects the various risks of the owners, which are very different under different circumstances. As the values of such properties are much affected by these neglected risks, his calculations of values are necessarily unreliable. He also constantly fails to distinguish between the widely different values under different laws and preponderant ideas of justice.

If a regulation were established which would follow consistently the principle of a uniform return on investments, without the various exceptions which the author is obliged to make in order to avoid absurdity, then the values would be in proportion to the investments;

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but entirely different laws govern values under free competition, under natural unregulated monopolies, and under any other kind of regulation. When one economic system is abandoned and another is introduced which produces entirely different values, the only just way of ordering the transition is to give to the owners of preceding enterprises the values which have arisen under the old system, or to offer them securities which they are willing to accept as equivalent.

The author in this and his previous paper, "The Appraisal of Public Service Properties as a Basis for the Regulation of Rates",* to which the present paper is a supplement, has Californian laws and conditions mainly in view, and considers principally those utilities for which public ownership is the only practicable method of regulation; but he does not limit the application of his theories to such cases.

He may object, to the method advocated by the writer, that for new properties it is impracticable, as the future revenues which a new property will be able to secure with the present regulation, cannot be estimated, and that the actual cost of such a new property is, therefore, the best available measure of its value. With a regulation adopting the principles here and previously advocated by the writer, which proposes to secure equal average profits in monopolized and competitive industries, it is not necessary to know the value, but only the investment, in new enterprises.

The profit of an enterprise is the annual return on the capital invested. When a new regulation is introduced, the owners of old enterprises must be given the values that have arisen under the old regulation, therefore these values must be considered as the investments in the old enterprises; and, for new enterprises, the actual investment must be taken. The annual net earnings of a monopolized industry must be compared with the investment thus ascertained in order to obtain the average profit of this industry. This average profit should be the same as the average profit of competitive industries of about the same risk.

It will not be practicable to ascertain the average profit in private firms and partnerships, but the average profits of stock companies engaged in competitive enterprises can be ascertained with the proper regulation necessary to secure the interests of minority stockholders. These enterprises are most nearly like public service properties, and, therefore, are best used to determine the average profit to be allowed to monopolized industries. After the investments in monopolized enterprises have been thus ascertained, they can be correctly kept up to date by a moderate amount of supervision.

For this purpose the author's definitions of Investment and Depreciation must be modified. The aim of this method is to secure in the

* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 828.

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future equal average profits on the actually invested capital of competitive and monopolized industries.

The proper definition for Investment, therefore, is: The capital invested is the aggregate of the actual expenditures which have been made in order to render the property in question efficient for the purpose for which it is intended. The attempt to find the degree of reasonableness of the investment, which requires an impracticable amount of supervision, should be abandoned.

When an article discarded is replaced by another, the cost of the new article should be added, and the original cost of the old one should be subtracted from the investment. Changes in land values do not affect the investment as defined; therefore, it is only necessary, in order to keep an investment up to date, to see that when renewals are made the cost of the new parts less the original cost of the renewed parts is added to the investment. The actual amounts of new capital paid in for enlargements of old and the creation of new enterprises must also be added to the investment. Adequate supervision to secure honesty is required.

In the competitive enterprises which are used to determine their average profits, the same principles of ascertaining the investments and keeping their amount up to date must be followed. For this regulation, one valuation for every old enterprise is considered, and determinations of the investments in new enterprises are required. The author recommends what he calls the "unlimited-life method" of providing for depreciation. The "actual-life method" would more accurately describe it. It is a great improvement over the extremely artificial methods, not in agreement with actual depreciation, which are often advocated.

To some extent, however, his method is marred by his implied assumption, that the usefulness or serviceability of an article remains the same from the beginning to the end of its life as a part of the enterprise. Usually, this is not the case; his estimates of depreciation are thereby falsified. The consequent error is opposite to that resulting from his neglect of interest as a factor in depreciation, so that the two errors partly balance each other, and his estimates may thereby, by chance, sometimes become correct ones. In his view, the only cause of depreciation is the approaching obligation to renew, and the loss of usefulness and the increasing cost of maintenance of many parts of an enterprise are additional causes of depreciation. Maintenance may be defined as the replacement of parts, and renewal as the replacement of wholes. They can only be defined and distinguished by defining what is a part and what a whole. The various parts of every whole have different lives. In a new article few parts soon reach the end of their life; as the article grows old, more and more do so.

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The cost of maintenance of most articles, therefore, increases with age, and their net usefulness decreases to the same extent. For calculating the real net earnings of any enterprise, the determination of the real depreciation of its parts is important, and the author's discussion, though marred by errors, is, nevertheless, valuable, as it combats successfully and exposes many widespread errors and erroneous methods based thereon. The just and economical regulation of the natural monopolies under private ownership requires definite contracts, between the public and the owners, which secure efficient management without excessive supervision, by giving to the owners compensation in proportion to efficiency, and which make the returns on invested capital practically the same in competitive and monopolized industries. This requires prices varying with the material costs of production.

This regulation is practicable where the material causes of differences in cost of production and their effect on it can be estimated accurately. It can be introduced successfully, also, in semi-competitive industries like interstate railways. In these, part of the business is competitive, and part, mostly the local business, is not subject to competition. Competition, as modified by control, prescribes equality of rates for equal quality and quantity of service to the buyer by different competitors, and differences of rates for different services in proportion to the value of the service to the buyer; but it does not prescribe the absolute size of the rates. Just rates, which comply with all the requirements imposed by competition and vary with material costs of production, can be fixed. To comply with the requirements of competition it will be necessary to make the profits of each railway on competitive business what is obtainable, and to even up things by allowing different profits on non-competitive business. The rates for each road must be determined so that its total earnings are the same as would result from the rates proper if there were no competition.

In determining the local freight and passenger rates of old companies, average profits on their values, as previously determined, must be considered as a part of the costs of production of all the services rendered by them. For new railroads, which are built while the new system of rate regulation is in force, the proper local rates cannot be determined without knowing their value, and this cannot be found from their revenue; therefore, it must be assumed to be equal to the cost of production.

When the public allows an increase of earnings to the railroads of an economic district in proportion to the new investments, it is vitally interested to prevent unprofitable ones. Therefore it must forbid the building of unprofitable roads, or the adoption of uneconomical plans for those it allows, by requiring previous approval of

Mr. Mayer. all new railroads by the public authority and by watching the expenses. This is far different from accepting the cost of production of enterprises built without public supervision as their value, and allowing an average return on it.

The present ignorance regarding the principles which will govern the valuation and future rate regulation of interstate railroads is a great evil, increasing the risk of investment and consequently the cost of the capital needed for their further development. An authoritative utterance of these principles can be brought about by intelligent public discussion of these questions. This, therefore, is very desirable.

The author, by describing clearly and endeavoring to improve a bad method of monopoly control, shows up its defects and will thereby hasten its abandonment.

The foregoing criticisms are largely directed against this bad method, and not against the author's very able and lucid description of it. His paper clears up much confusion; it combats successfully many current errors, and suggests the correct solution of such problems as are not completely solved by him. The paper, therefore, is valuable, and well worth careful study; the author has rendered a great service by its presentation.

Mr. Green. F. W. Green,* Assoc. M. Am. Soc. C. E. (by letter).—Of late, one might say that the subject of depreciation has become the very apotheosis of platitudinous piffle, on the part of some writers, of whom it may regretfully be said, *"damnant quod non intelligunt"*. In the present instance, however, the author has handled his subject most commendably, and has shown a thorough knowledge of its fundamentals, which could only have been the result of studious application and long experience. He has covered the subject so thoroughly that very little else need be said.

The recent papers before this Society by William J. Wilgus† and John W. Alvord,‡ Members, Am. Soc. C. E., and the present paper by Mr. Grunsky, will undoubtedly become milestones in the development of the subject of valuation; and the latter paper happily supplements the two former, both as to logical presentation, and clear enunciation of principles.

It might not be amiss, here, to give the following definitions:

"Value. Specif.: *Economics.* a—Efficiency in exchange; power which an object confers upon its possessor, irrespective of political compulsion or personal sentiment, to command the commodities and services of others; purchasing power in the abstract. b—Concrete

* Stamps, Ark.

† "Physical Valuation of Railroads," *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 203.

‡ "The Depreciation of Public Utility Properties as Affecting Their Valuation and Fair Return," *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 788.

purchasing power; the specific quantity of another object for which a given object can be exchanged; a price which can be actually obtained. The value of an article depends not upon its *total* utility, but upon its *marginal* utility (see under Utility), diminishing as the supply increases. It will usually be proportionate to the cost of production, because when the value of an article is above its cost producers will tend to increase the supply, while if its value is below its cost producers will tend to diminish the supply. c—Proper price; the quantity of money, goods, or services which an article is likely to command in the long run, as distinct from its price in an individual instance; a legitimate price, as distinct from an unfair or extortionate one;—sometimes called *normal value,* in contrast to *market value.* 'The commercial or competitive theory bases *value* upon what the buyer is willing and able to offer for an article; the socialistic theory bases it upon what the article has cost the seller in the way of toil and sacrifice.'—*A. T. Hadley.* d—The estimate which an individual places upon some of his possessions as compared with others, independently of any intent to sell;—sometimes called *subjective value,* or, less correctly, *value in use,* and employed in a loose sense, as nearly equivalent to *utility. Value* in use is utility and nothing else, and in political economy should be called by that name and no other.—*F. A. Walker.* Mr. Green.

"Utility. Power to satisfy human wants. 'Wants are here reckoned quantitatively with regard to their volume and intensity; not according to any ethical or prudential standard' (*A. Marshall*). Modern economists distinguish the *total utility* measured by the loss which would be involved in the entire deprivation of a commodity, from the *marginal utility,* or *final utility,* which would be involved in the loss of a small amount of it. Thus, the *total utility* of water is infinitely great; but the *marginal utility* of a single quart of water is ordinarily very small, because a man or a community has usually so much water at command that one quart more or less makes very little difference.

"Depreciation. Act of depreciating, or state of being depreciated; specif.: a—A falling of value; of money, a reduction or loss in exchange value or purchasing power, esp. with reference to the face value. b—A lowering in estimation; disparagement."

The foregoing are from Webster's New International Dictionary. The following is from the Century Dictionary:

"Depreciation. 1. The act of lessening or bringing down price or value. 2. A fall in value; reduction of worth. 3. A belittling or running down of value or merit; conscious undervaluation or underestimation of the merits of a person, action, or thing; unfavorable judgment or scant praise."

Perhaps some of the confusion extant, as to depreciation, in a valuation sense, has been due to some writers following Definition 3 through misapprehension.

In the 11th edition of the Encyclopedia Britannica, p. 867 *et seq.*, Vol. XXVII, under "Value", will be found some interesting information; also in Dr. Taussig's "Principles of Economics," Vol. II, pp. 363-418, incl.; both are commended to the diligence of the uninformed.

Mr. Green.

A booklet, entitled "Depreciation of Railroad Property", by Richard J. McCarty, M. Am. Soc. C. E., is a very valuable contribution to the literature of this subject.

In the various discussions before this Society, a considerable proportion of those who have submitted their views, obviously confound physical depreciation with functional depreciation; hence a few remarks may perhaps be apropos.

It is equally to the interest of the rate-payer and the utility that operating expenses, taxes, and interest, should be at such a minimum as is consistent with economy and good service. Depreciation is legitimately a part of operating expenses, and its sole function is to maintain, unimpaired, the original capital investment. (It would be silly to say that this depreciation fund must be in specie, and conspicuously displaced by the utility in a glass case, to the end that the rate-payers might be well assured, as often as desired, that the utility was actually possessed of the necessary funds to maintain unimpaired service. If the utility could not be freed from the imputation of truculence, it should never have been permitted a corporate birth.) The measure of economy is the requirement that the annual cost of an article shall be a minimum. Annual cost is the sum of interest, maintenence, and depreciation. Hence, the rate-payer profits in every case where the utility follows the principle of minimum annual cost. It is against the interests of the rate-payer, as well as those of the utility, to base rates on depreciated physical value. The classic illustration of the railroad cross-tie will make this clear. Suppose it were possible for a railroad to purchase a tie of such lasting qualities that it would have a serviceable life of 50 years. This tie will function at 100% efficiency for 50 years, and it is to the rate-payer's interest that it serve its functional life before taken out of the track. At the end of 25 years, however, the utility will be entitled to receive, the premises considered, a rate of return based on a 50% physical valuation. There is no incentive, therefore, for the utility to get the full service out of the tie, but, on the contrary, the sooner it is renewed, the higher the value on which rates are presumed to be based. In such a conflict of interest, it is but natural to conclude that the rate-payer will be required to pay rates based on extravagant maintenance, which has been forced on the utility by the rate-payer; the latter, indeed, has deducted what some have euphemistically styled "the return of capital to the owner", but, in so doing, has quite deservedly been beaten at his own game. In such case, the sum of interest, maintenance, and depreciation would hardly be a minimum; hence, the economy which should inure to the benefit of both the utility and the rate-payer is non-existent. It is *in posse*, but, in the case assumed, not *in esse*. The conclusion, therefore, seems inevitable that, if the interests of both the utility and the rate-payer are properly to be conserved, valuation should comprehend functional depreciation, but should exclude physical depreciation.

RICHARD J. McCARTY,* M. AM. SOC. C. E. (by letter).—This paper seems to be a long step in the right direction toward clearing the way of many of the obscurities which envelop the question of valuation, and it is hoped that it will be carefully read and given the attention it deserves by all who are interested in that important subject. Mr. McCarty.

Mr. Grunsky denominates his method of making appraisal as the "Unlimited-Life Method", but it would seem that, with equal propriety, it might be called the "Common-Sense Method", particularly with respect to the method of dealing with depreciation.

All theories of valuation which involve the deduction of depreciation from cost of reproduction in the determination of what is known as "fair value" fail to take into consideration certain important practical features involved.

There are three possible bases on which to estimate the cost of reproduction of physical property of a public utility:

1.—The cost, by present methods, and under present conditions, of an equivalent property capable of rendering the same service as the existing property;

2.—The cost, by present methods, and under present conditions, of a property identical with the existing property; and

3.—The cost of the existing property, at present prices, with the methods and conditions under which the property was created, developed, and brought to its existing state.

The theory of cost of reproduction of physical property is based on two principles, each of which is sanctioned by commercial custom, is in accord with accepted theories of economics, and is justified by the Court decisions that will be quoted under each.

The first of these principles is: The investors are entitled to include in "fair value" the proper original cost or all values honestly and judiciously sacrificed in producing the property.

This principle is justified by the Supreme Court of the United States in Reagan *vs.* Farmers Loan and Trust Company, thus:

"And yet justice demands that every one should receive some compensation for the use of his money or property, if it be possible, without prejudice to the rights of others." (154 U. S. Sup. Ct., 362.)

Here, surely, "some compensation for the use of his money or property" cannot be taken as meaning less than a fair rate of return on all money or property actually invested in good faith and with good judgment in a property which is able to justify its existence, and manifestly such a return for such a property cannot prejudice the rights of others.

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* Kansas City, Mo.

Mr. McCarty.

The principle, however, was expressly affirmed by the Court of Appeals of New York, on March 24th, 1914, in People *ex rel.* Kings County Lighting Company *vs.* Willcox *et al.,* as follows:

"The right to limit the corporation to a fair return fixed by public authority necessarily involves the correlative right in the corporation to be assured of that fair return during all the time that its capital is employed in the public service." (N. E. Rep., May 5th, 1914.)

The second principle is: The investors are entitled to any appreciation due to increased prices of land, labor, or materials invested in the property.

This principle is justified by the Supreme Court of the United States in Willcox *vs.* Consolidated Gas Company thus:

"And we concur with the court below in holding that the value of the property is to be determined as of the time when the inquiry is made concerning rates. If the property which legally enters into the consideration of rates has increased in value since it was acquired, the company is entitled to such increase." (212 U. S., 52.)

Here, "the property which legally enters into the consideration of rates", naturally includes all land, labor, and materials invested in the property.

The Cost, by Present Methods, and Under Present Conditions, of an Equivalent Property Capable of Rendering the Same Service as the Existing Property.—An equivalent plant, as a rule, will involve either more or less land, labor, and materials than has been invested in the existing plant. If more, then the investors would be receiving the benefit of physical property they did not invest, which cannot be justified. If less, then the investors would be deprived of a certain portion of the investments actually made, and the first principle set forth would be violated.

Again, under the cost of reproduction of an equivalent plant, the investors either would secure increased prices of land, labor, and materials which had never been invested, which is unfair, or would be subjected to decreased prices of land, labor, and materials which they had actually invested in the property, which is in violation of the second principle.

The Cost, by Present Methods, and Under Present Conditions, of a Property Identical with the Existing Property.—In an identical property the land and raw materials would be practically the same as those involved in the existing property, and the cost of reproduction of these items would be no violation of the principles mentioned, except in so far as the prices would be less than those originally paid, in which case the cost of reproduction would violate the first principle set forth, because it would subject the investors to a deduction from the proper original cost of the property.

Mr. McCarty.

Again, in an identical property, built by present methods, the amount of labor, except by accident, would be either greater or less than in the existing property. If greater, the investors would get the benefit of labor they did not invest, which cannot be justified. If less, they would be deprived of a certain portion of labor they did invest, and the first principle would be violated.

Moreover, should the reproduction labor be greater than the invested labor, the investors might get increased prices of labor not invested, which is unfair. If the reproduction labor should be less than the invested labor, the investors might be denied increased prices on certain labor which they did invest, and the second principle would be violated.

The Cost of the Existing Property, at Present Prices, with the Methods and Conditions Under Which the Property was Created, Developed, and Brought to its Existing State.—This method of estimating the cost of reproduction gives the identical amount of land, labor, and raw materials used in the existing plant at current prices, and complies with both of the principles set forth, except in case the present prices should be less than the prices actually paid in creating and developing the existing property. In that event, the investors would be subjected to a deduction from proper original cost of their property, which is in violation of the first principle set forth.

In view of all these considerations, it seems clear that,

1.—The proper basis for estimating the cost of reproduction is the cost of a property identical with the existing property, at present prices, with the methods and conditions under which the existing property was developed.

2.—In estimating the cost of reproduction, the items should be determined so that those which are greater than the original cost can be separated from those which are less.

3.—In using cost of reproduction for determining "fair value", only those items are fairly available which are equal to or greater than the corresponding items of proper original cost.

Evidently, therefore, the general theory of cost of reproduction which subjects the investors of a public utility to a loss, because of decreased prices of land, labor, and materials, cannot be justified, because it violates the recognized principle that they are entitled to a fair return on their capital during all the time that it is used in the public service.

It will be held, of course, that, if they are to receive the benefit of an increase in prices, they should stand the losses incident to a decrease. This, of course, is true of private investors, who have the right to dispose of their property to the best advantage at any given time and recover their losses by other investments. Investors in

Mr. McCarty.

public utilities have no such right. Therefore, they should not be subjected to the consequences of a right that they do not enjoy.

However, even were the cost of reproduction determined by making proper allowance for all decreases in price, the deduction on account of depreciation cannot be justified. For instance:

Suppose a new railroad, from the time it commenced operation, should charge to its operating expenses an amount sufficient to take care of the depreciation which begins in each perishable part at the moment of its installation, and to provide a fund for renewing that part when it shall have been worn out.

In the course of time the property will settle down to a condition at which the depreciation of the whole will be an average of from 10 to 25% of the original cost. This depreciation in no way affects the safety and efficiency of the property, and cannot be made good until the parts reach the limit of their usefulness, without a waste of serviceable material.

In case the original cost should be, for illustration, say, \$10 000 000, there would at that time, and in the event the property had been able to earn it, a fund of from \$1 000 000 to \$2 500 000 collected in the way of rates and set aside to take care of this accrued depreciation; but, as the depreciation cannot be made good without an undue waste of serviceable material, this fund could not be applied to the purpose for which it had been created. This large sum would be paid through rates in excess of those actually necessary, and at a time when the public would be least able to pay, so that the impropriety of such a course would seem to be sufficiently clear.

The only justification for the deduction of this fund from the so-called "fair value" of the property would be its complete and unconditional restoration to the investors, that is, they must be put in precisely the same position with respect to it as before its investment in the railroad property. Therefore, if deducted from "fair value", it must be distributed among the security holders, not as dividends, but as return of capital. This would at once raise the question as to whether the security holders, having legally invested without any such condition, could be compelled to accept such payment, and, if so, whether the bondholders should participate in the distribution, and, if so, on what basis and in what manner could the distribution be made. Furthermore, it is well known that no man can determine the amount of depreciation in question, and that no two men seem able to agree on this point.

It seems clear, therefore, that the conclusion reached by Mr. Grunsky, to the effect that, in the appraisal of a public utility, all considerations of depreciation are unnecessary, is entirely sound.

Mr. Grunsky quotes the decision of the Supreme Court of the United States in Knoxville *vs.* Knoxville Water Company, as follows:

"If, however, a company fails to perform this plain duty and to exact sufficient returns to keep the investment unimpaired, whether this is the result of the unwarranted dividends upon over-issues of securities, or of omission to exact proper prices for the output, the fault is its own." Mr. McCarty.

As Mr. Grunsky states, the Court in this ruling assumes that the amount of profits of a public utility can be increased or decreased by increasing or decreasing the rates, and fails to recognize that an effective way of destroying the business of a public utility is by an undue increase in rates. The reason that this feature was overlooked by the Court is no doubt due to the fact that it was not properly developed by counsel at the trial of the case. This, in turn, was no doubt due to counsel not having been posted by the management in all the practical details of the situation.

This shows the importance, not only of establishing a correct theory of valuation, but of establishing it in such a way as to be perfectly clear to the legal mind, to the end that it shall be properly presented to the Courts.

PHILIP W. HENRY,* M. AM. SOC. C. E. (by letter).—Not always do engineers and accountants agree in matters relating to the keeping of accounts, but, in regard to the definition of depreciation, they are in very close accord. For instance, in Keister's "Corporation Accounting and Auditing" depreciation is defined as: "The actual loss upon assets which are diminishing in value or it is an estimated sum charged against gross revenue; which amount is considered sufficient to replace the capital used up or reduced by wear and tear"; and Bentley's "Science of Accounts" states that "Depreciation of a fixed asset indicates its probable decrease in value by reason of wear and tear, and the efflux of time". Mr. Henry.

For the definition of depreciation by engineers, we have that given by the author as "the lessening in worth of any perishable article or property". Annual depreciation is the "annual theoretical lessening in worth, expressed in dollars". Another definition is that given by Mr. Alvord† in his paper on "Fundamental Principles of Public Utility Valuation": "The lessened value of any property, structure, or machine, due either to its wear, loss of usefulness, growing lack of adaptation or approaching abandonment".

The definition, however, which, on account of its authoritative source, is controlling to individuals and to corporations of all kinds—public utility or otherwise—is that given by the Internal Revenue Bureau of the Treasury Department in connection with the income tax. It is interesting to note how close the agreement is between the definition given by the accountants and engineers already quoted and

* New York City.

† *Transactions*, Am. Soc. C. E., Vol. LXXIX, p. 117.

Mr. Henry. that of the Internal Revenue Bureau. On Form 1035, "Return of Annual Net Income", for Miscellaneous Corporations, Section 2, approved by Congress, on October 2d, 1913, a deduction from gross income is allowed for "Total Amount of Depreciation". This item is defined more particularly: "The amount claimed under Item 5 (for depreciation) should be such an amount as measures the loss which the corporation actually sustains during the year in the value of buildings, machinery and such other property as is subject to depreciation on account of wear and tear, exhaustion or obsolesence".

It is evident, therefore, that the confusion which exists in regard to depreciation is caused, not so much by disagreement on what depreciation really is, as to the manner of applying it to any given case. Therefore, although the principles governing depreciation are the same, whether applied to a mining property having a definite life time, or to a public utility corporation having an unlimited life time, because certain physical assets of both are subject to "probable decrease in value by reason of wear and tear, and the efflux of time", the method of applying these principles must necessarily be different. With both types of corporations, however, it is necessary to set up a depreciation reserve, and, in both cases, the amount in the depreciation reserve at any given time should equal original value less present value, using cost and value as equivalent terms and neglecting other elements (than depreciation) affecting valuation. The author brings out this principle when he states that "The investment less this obligation to replace (also less deferred maintenance) is, in other words, a measure of present worth."

From the definitions given by the author, it is evident that investment and original value are equivalent terms, and that obligation to replace plus deferred maintenance is nothing more than depreciation reserve. It seems unfortunate that the author should have used these terms instead of depreciation reserve, which has an accepted meaning among engineers and accountants, and is recognized by the Internal Revenue Bureau, as follows: "Where a depreciation reserve is set up, all renewals and replacements must be charged to such reserve, and the addition to this reserve each year must be a fair measure of the loss which the corporation sustains by reason of the depreciation of its property." Also "Where depreciation of physical property is made good by renewals, replacements, repairs, etc., and the expense of such renewals, replacements, repairs, etc., is charged to the general expense account, no deduction for depreciation can be made in the Return of Annual Net Income." This ruling indicates that depreciation reserve should be set up at the beginning of operation, and that it should truly represent "the loss which the corporation sustains by reason of the depreciation of its property".

In the principle of present value plus depreciation reserve equaling original value, it is assumed that original value and original cost are equivalent terms, whereas, in practice, it is known that the original cost may be considerably greater than original value owing to bad judgment or bad management; or, on the other hand, original cost may be less than original value through the purchase of a property at a forced sale. Mr. Henry.

It is evident also, as a principle, that although the money in the depreciation reserve may be invested either in outside securities or in the property itself for betterments and additions, it should not be available for the payment of dividends, but only for making replacements or for the repayment of capital.

It is also evident that the amount in the depreciation reserve will not necessarily increase each year, for, because of the large replacements being made in a given year, the amount in the depreciation reserve at the end of that year may be less than at the beginning. Present value, however, will be increased by the exact amount that the depreciation reserve is decreased.

With a depreciation reserve carried in accordance with the principles laid down, no difficulty arises as to whether a public utility corporation should receive the return authorized by some public service commission on its original value or on its remaining value, for, at all times, the property is kept up to its original value through the amount in the depreciation reserve. In buying such a property, the purchaser will be quite willing to pay the original value (cost), for the original value will be there in present value plus depreciation reserve. In buying a corporation which has not set aside a depreciation reserve, the purchaser will naturally pay only the amount represented by present value, but, at the same time, he must assume an obligation to put into the property, later, an amount equal to that which the corporation should have carried in a properly adjusted depreciation reserve.

As to the method of setting aside depreciation reserve and its treatment on the books, the writer has already given his views in his discussion on "Valuation for Rate-Making Purposes."*

CLINTON S. BURNS,† M. AM. SOC. C. E. (by letter).—The writer is particularly interested in expert valuation work and appraisal of public service properties, and is glad to express his appreciation of this valuable and interesting paper. Mr. Burns.

In his discussion of a paper‡ by John W. Alvord, M. Am. Soc. C. E., the writer presented his views on this subject somewhat in detail, and will now merely amplify certain phases of that discussion.

* *Proceedings*, Am. Soc. C. E., for April, 1914.

† Kansas City, Mo.

‡ "The Depreciation of Public Utility Properties as Affecting Their Valuation and Fair Return", *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 842.

Mr. Burns.

If the expert appraiser could place himself in a position of absolute impartiality in the consideration of this question, so that his conclusions would be entirely unbiased by any consideration of favoritism toward his client, he should then feel satisfied with the results of his analysis, and the writer would suggest that this result could be obtained by assuming, for the purpose of this discussion, that there are no private interests at stake and that the property being valued is municipally owned, having been built by a bond issue bearing 6% interest and having been operated heretofore (as most municipal plants are) without any particular reference to proper rates or adequate returns. Now there comes a business-like municipal administration requiring that an appraisal be made of this municipal plant, together with an investigation of its revenues, this appraisal to be used as a basis for a revision of the rates, the object being that the new rates shall produce adequate revenues to make the property self-supporting.

Let it be assumed that the appraiser finds the property described in fairly good operating condition, and proceeds to determine how much operating revenue must be collected in order to preserve the property in satisfactory operating condition and perpetuate the utility. It is obvious that the fundamental principles involved in such an investigation are entirely independent of whether the property is owned by a municipality, by an individual, or by a corporation, and these principles are the same for a short-lived property as for one of long life; and it may simplify the problem and clarify the analysis if it be assumed that the property is made up entirely of perishable items, the entire property having the same average life or expectancy as that for which the original construction bonds were issued. In these assumptions, it is evident that the property is made just self-supporting when the revenues are sufficient to pay all operating expenses—6% interest on the bond issue and a sufficient sum to renew the plant when the original bonds become due, this being the expectancy of its life. It is evident, therefore, that the renewal fund, or replacement fund as it is termed by the author, must be sufficient to retire the original bond issue plus whatever sum may have to be accumulated to pay for the increased cost of renewal of the property at the expiration of its life, in case such renewal costs more at that time than the original bond issue; or, conversely, minus whatever sum the renewal may cost less than the original bond issue. The fund that will retire the original bond issue, plus or minus the corrections necessary to care for changed conditions in the cost of renewal, therefore, will represent the proper replacement fund in this instance. This view of the problem makes it plain that, in this case, no matter what the present value or the depreciated value of the property may be, the rates must be based on the future cost of reproduction, else, when

the time comes that replacement is necessary, it will be found that the revenues have been inadequate to provide for replacement and, therefore, the property has not been self-sustained. In the case of a municipal plant, as assumed in this analysis, the emergency is met by a larger bond issue, but this is not an equitable solution, because it is discriminating against the future generation in favor of the present. Mr. Burns.

This analysis* also throws light on the equity of including the cost of cutting and replacing existing street pavements, where the object of the appraisal is to determine a basis for rate-making. It has been the practice of many State Public Service Commissions to adhere in part to the theory of the cost of reproduction of a property, but to eliminate from such reproduction cost such items as the added cost due to street pavements, on the theory that this added cost, not having been borne by the utility and therefore not constituting a portion of its investment, is properly eliminated. The fallacy of this argument has been pointed out so frequently and so forcibly by the leading expert appraisers that it may seem superfluous to call attention to it again in this discussion, but the persistency with which some of the State Commissions adhere to these fallacies is sufficient excuse for the reiteration.

The fallacy is that such a procedure confuses present value with past cost. However, irrespective of that fact, it occurs to the writer that all existing pavements must be included in any valuation that is to be used as a basis for rate-making, because this determines to what extent the owner of the utility will be compelled to provide funds with which to restore the pavements when he is actually confronted with the question of renewing the obsolete or decayed pipes. The following will perhaps emphasize this truth.

A public utility, as stated by the author, has unlimited life; it must be perpetuated; therefore, every perishable part must be renewed by the owner whenever such part becomes incapacitated for its work. Every pipe line must ultimately meet this fate. The revenue must be sufficient to pay all operating expenses, interest on the investment, together with an additional sum sufficient to perpetuate the property. To perpetuate means not only to provide for the ordinary maintenance, repairs, and up-keep, but to guarantee the requisite replacement fund as well. Now, every street that is paved necessarily increases the owner's expenses in renewing the pipes, and such renewal is a liability which he must ultimately meet face to face. Therefore, if present rates are based on a valuation that does not include the element of enhanced value accruing from the item of street pavements, then the revenue is deficient and confiscatory by just that amount. Similarly, all metropolitan influences, such as growth of traffic, interference of sewers, telephone conduits, underground electric systems, police regu-

Mr. Burns. lation, street railways, etc., each of which tends to increase the difficulty and cost of renewal of structures—particularly of pipe systems and other underground structures—must be included in any valuation to be used as a basis for rate regulation, for if any of these be not thus included, then, when the time comes for replacing the old structures, additional capital must be incorporated in the property, with no corresponding betterment in the construction account.

Concerning the author's statement that "the annual replacement increment * * * should, * * * ordinarily be assumed to be earning interest at the same rate as the entire business. * * *, if the same be invested in large part in the business", the writer wishes to direct attention to the fact that such re-investment of the annual increments of the replacement fund entitles it to a higher rate of interest than is justifiable in computing the sinking-fund accrument; or, to state it another way, the 7% rate of return, earned by the waterworks or gas plant referred to by the author, is properly considered as not only the earnings of the invested capital, but includes also the owner's risk, his reward for industry, his time devoted to the investment, and all the other elements which go to make up the difference between liquid and invested capital. It seems to the writer, therefore, that the proper rate of interest to be used in all sinking-fund computations is the current rate for liquid capital, regardless of how much the invested capital may be earning.

The author states that the correctness of any method of appraisal for rate-fixing purposes is dependent on its continuous operation from the beginning. It seems to the writer that although this may be true, yet therein lies the real difficulty. It is a practical impossibility to attain this ideal condition of affairs. The beginning is past and gone, and the future is uncertain. Many public utility corporations have been content in the past, or in the early part of their development period, with revenues that were far from adequate, simply because of their hope and their faith that the future would enable them to operate on revenues sufficiently remunerative to compensate for the present; but the usual situation is that when the future arrives and their hopes are beginning to be realized, there comes the agitation for rate regulation and reduction in the revenues. The fact is that the real question at issue is to determine the starting point, and what is a fair revenue for the particular property under consideration at the present time and in the future. Surely there must be some means to determine what is a fair revenue for a property without reference to its past history, for, in a large majority of public service properties, there is no reliable past history from which sufficient data can be secured to serve as a basis of equity in the future. Furthermore, what may have transpired in the past cannot in equity be taken into consideration in fixing rates for the future, because it is evident that

the sole purpose of rate regulation is to secure justice between the public utility and its patrons. The public in many cases has not been satisfied, either with the service or the charges made heretofore, and it is to secure equity in the future that the necessity arises for investigating public utility corporations. It is apparent that no past inequalities can be taken into consideration in fixing a rate for the future, because, in many cases, the present owners of the public utilities have not been the owners during the time when the inequalities may have been the greatest. Furthermore, the public of to-day and of the future are not the same individuals with whom the corporation has had its past dealings. Therefore, the fact that the rates may have been insufficient in the past, either in the early history of the corporation or throughout its entire life, cannot be considered as justification for fixing future rates on a higher plane than that of equity between the corporation and the rate-payers, as determined by the present value of the property; neither can the fact that the rates in the past may have been more than adequate be considered at the present time as a justification for reducing them for the future below this same plane of equity, based on the present valuation. It is evident, therefore, that to introduce the factor of liberality in fixing rates for the future to compensate for past insufficiencies in return, or *vice versa,* would be an exceedingly dangerous precedent for a commission to establish in striving for equity in the solution of public utility problems; or, in other words, the law is neither for charity nor for vengeance, but for equity. Mr. Burns.

In conclusion, the writer submits that the prime difficulty in fixing any system of rates is to secure an equitable starting point; but, as a general rule, the nearest approach to fairness and equity to all interests may be secured theoretically by ignoring the past history of the property and taking a new start, using the undepreciated cost of reproduction as the basis of all computations and the full life terms of each item of the property in the computation of the replacement fund, on the sinking-fund basis, and safeguarding the public by proper means which will guarantee that the utility will maintain the efficiency of the property consistently with the unlimited-life theory. In many cases, however, this theoretical condition cannot be attained owing to State laws that impose certain limitations upon the treatment of the question, or, owing to the interpretation of the law in accordance with the individual notions of certain commissions, and many other influences which the appraiser must recognize in the practical presentation of his problem. It is not meant by this statement that an appraiser should allow any State commission to deflect his judgment from what he believes to be the right results, but rather to convey the idea that it may be futile to present a subject in direct

Mr. Burns. opposition to the rules and regulations of the commission, even though those rules are improper and unwarranted.

In many cases it is more practical, therefore, to compute the depreciation of the property and use the depreciated value as the basis of rates, keeping in mind the fact that the average public utility cannot attain a depreciation of more than approximately 30 or 35%, and seldom does exceed one of from 10 to 15 per cent.

Take, for example, a water-works system, the average expectancy of which will ordinarily be, say, 60 years. The maximum depreciation that it can attain and remain in operating condition is reached after it passes the age of complete renewal of all its parts and is living its second life. When it reaches this state, it has an average age of 30 years, an expectancy of 30 years, and, with liquid capital at 3% interest, the entire property has a depreciation of 29 per cent. This may be termed the ultimate limit of depreciation of this property, and as this ultimate limit can never be reached in any growing property, and usually seldom exceeds 15% of the cost of reproduction, it is not vital whether the reproduction cost of the depreciated value be used, in view of the fact that the replacement allowance must be greater in the latter case than in the former.

In reviewing a list of appraisals of public utilities valued by the writer, comprising water-works, electric plants, gas works, and telephone systems, aggregating a reproduction cost of \$21 329 494, it is found that the aggregate depreciation for this entire list is \$2 833 441, or an average of approximately 13%, and in using the depreciated value in each of these properties, it is found that but little difference is reflected into the rates, as compared with what would have resulted from the perhaps more theoretically correct method of using undepreciated reproduction cost.

Mr. Gillette. HALBERT P. GILLETTE,* M. AM. SOC. C. E. (by letter).—Depreciation is loss of value. It is not loss of cost. There is depreciated value, but there is no such thing as depreciated cost. Accrued depreciation, therefore, must be associated with value if it is to have any real significance.

The value of a productive property is the present worth of its prospective net earnings, or, as commonly expressed, its "capitalized net earnings". In reference to plants, the term "value" can be used in no other sense and still retain its commonly accepted meaning.

Any measure of accrued depreciation must necessarily be a measure of value. The only criterion by which we can measure correctly the value of an old plant unit is a new plant unit capable of giving the same service in the most economic manner.

* Chicago, Ill.

We have this general rule for determining the value of an old plant unit: Assuming gross income to be constant, the depreciated value of an old plant unit is equal to the first cost of a new plant unit of most economic design minus the capitalized difference in their equated annual operating expenses during the prospective economic life. Mr. Gillette.

Operating expenses in this rule must include repairs and taxes, but must exclude functional, that is, commercial, depreciation and interest charges. The interest rate used in "capitalizing" must include both the normal rate of return on capital and a percentage sufficient to provide sinking-fund annuities that will retire the plant unit at the end of its prospective functional life. The annual operating expenses must be equated thus: To equate operating expenses, calculate the total cost of the operating expense of each year at compound interest up to the end of the last year of economic life of the plant unit. Add these compounded costs together and multiply by the annual deposit in a sinking fund which will redeem $1 at the end of the economic life. The product is the equated annual operating expense.

When the rule for depreciated value, previously given, yields a value equal to or less than the salvage value of any plant unit, that plant unit should be retired, because it is no longer economic. The rule, in practice, must be applied to the various units or groups of similar units of equal age that constitute the plant. There is no other rule or method that yields a theoretically correct depreciated value.

The depreciated value of a plant, derived by this rule, or by any other rule that approximates its results, must never be taken alone as a base for rate-making, for this depreciated value of the plant is only part of the total value of the property. The total value of the property, of which the plant is a part, is the present worth of its prospective net earnings. Hence, depreciated value is only part of the total value of which the other part is capitalized net profits. Thus, the total property value is $v = a + b$.

In this equation, a is the depreciated value of the plant and b is the remaining value of the property, or the non-physical value. If b is eliminated from consideration, then a has no useful significance in a rate case. Perhaps this fact is best illustrated by the value of city real estate, that is, land plus buildings. The value of the land is the capitalized net ground rent, which is deduced by subtracting the depreciated value of the building from the capitalized prospective net rent of the entire property. Eliminate the ground rent from consideration and the entire valuation problem vanishes, for the value of the building depends on its association with valuable ground. Similarly, as to a plant and the business that comes with its use, if the business is eliminated, the plant ceases to have more than salvage value.

Mr. Gillette.

In the sophistical reasoning so common in rate decisions, it is held that $v = a +$, etc. Then the "etc." is either forgotten or is arrived at by a cost method, for example, development cost by the deficit method. This is not reasoning; it is, at best, compromising; but, it is urged, commercial value cannot be a basis for rates, for that involves circular reasoning. If this is granted, then it follows logically that no commercial value of any kind can enter the rate-making problem. And as depreciated value is the commercial value of a plant, depreciated value, according to this hypothesis, can no more be used in a rate-making problem than can any other sort of value.

The units of an old plant unquestionably have less value than if they were new; but, conversely, the non-physical element has a greater value in an old property than in a new one, other conditions being equal.

The fundamental injustice in the ordinary rate-case decision lies in recognizing the loss of value of plant occurring with age, while refusing to recognize the co-existent increment in non-physical value also occurring with age.

Depreciated value must always be associated with capitalized net profits if there is to be a logical interpretation of the value of a productive property.

From another point of view, it may be seen that accrued depreciation cannot be logically considered apart from prospective net earnings. Let a commission enforce the rule that depreciated value is a base for rate-making. It would then follow that no astute public service company would undertake to improve its own plant by invention of new plant units, for to invent a new unit of a given class would result in depreciating all the existing units of the same class. An appraisal just prior to the replacement of the old units by the newly invented units would give the former only scrap value in such a case. An appraisal immediately following the replacement of old by new units would give a return on the undepreciated value of the new units, it is true, but would exclude all the old units that had been scrapped. In either case, the company would lose the amount of the accrued depreciation of the old units, and the loss would be consequent on the company's own efforts to reduce operating costs. Thus we reduce to an absurdity the proposition that rates should be based on depreciated value where depreciation is the result of invention.

The same sort of reasoning applies to depreciation due to economic inadequacy. Instead of putting in a small number of large plant units and replacing a larger number of small units, a utility or railway company would find it more profitable to avoid the functional depreciation thus caused, and it would add more small units to its plant. The result would be an uneconomic plant, viewed as we now view economy; but it would be economic to the company, viewed as

Mr. Gillette.

it is proposed to view depreciated value as a basis for rates. Under such conditions, a public service company or railway would be foolish to encourage invention or other action to change existing plant units. Not only would it discourage its own engineers and other employees, but it would frown on the efforts of outsiders to effect plant improvements. If forced by a commission to install a new plant unit, the company would endeavor to keep all its old plant units in "use", if only as standby equipment, so as to secure a return on the old investment.

This *reductio ad absurdum* serves to illustrate clearly the fallacy of basing rates on depreciated value, for it is apparent that depreciated value of plant units has no true economic significance save in connection with the value of the net earnings of the entire property. Rates might be based on actual cost of plant, and still yield an economic absurdity, but to base rates on depreciated value is to pile the absurdity even higher. Because of the distinctions between cost and value, between the agency theory and the competitive theory, and between present and past standards of equity, it is difficult to avoid logical pitfalls in a rate case. Many engineers are badly confused as to appraisal principles, so it is not to be wondered that commissions and Courts still err, not only as to depreciation, but as to many other matters. Let us, however, not deny the existence of accrued depreciation; but let us point out that depreciated value cannot logically, or fairly, or economically, be taken as a base on which to calculate the total "fair return" to investors in railways and public utilities.

Mr. Kiersted.

W. Kiersted,* M. Am. Soc. C. E. (by letter).—The author of this paper has taken pains to define carefully the various terms used in his very interesting discussion of the question of depreciation as an element to be considered in the valuation of public utility properties; but, notwithstanding all the care thus bestowed on the definition of terms, it is a question whether the atmosphere is altogether cleared of confusion.

For instance, in the definition of "fair value", the author states:

"The term 'fair value' as used in connection with rate-fixing is difficult to define. * * *

"The value to an investor is unhesitatingly determined from the net earnings, with due regard to hazards of the business. The value for rate-fixing purposes is to be that value on which, with the same regard for the hazards of the business, the owner is to be allowed to earn a fair interest return. Value should be the same whether determined by a rate-fixing body or whether determined for a purchaser. 'Fair value', then, is necessarily based on proper and reasonable investment which may be ascertained from cost records, and, when cost

* Kansas City, Mo.

Mr. Kiersted.

records are not available, is usually estimated by the 'cost-of-reproduction' method."

Confining the remarks of the writer to public utilities of a monopolistic character, like municipal water-works, lighting plants, and other properties of a similar character, it is clear at the outstart that, as the author states, value to an investor in any of these public utilities is determined from the net earnings. It is not so clear that net earnings should invariably provide a fixed rate of return on money invested in property actually in use, for this depends on social and industrial conditions, whether retrograding, stable, or improving. In any event, value, being altogether dependent on earning capacity, is an element of consideration only after a public service commission shall have concluded its labor and definitely fixed a rate of return for the property. To regard value as an element of consideration before the conclusion of a commission's work is putting the cart before the horse. Investment and value are not synonymous terms.

The work of a commission possesses really two independent functions: one is the determination of a basis for rate-making; the other is the process of rate-making. Value is the result of this twofold operation. If the public service commission fixes a rate of return below that which will invite investments in a particular kind of public utility, capital is deterred from investment and value is depreciated; on the other hand, if the rate of return is equal to or above an inviting rate, no trouble is likely to arise in securing capital for the improvement and extension of the property as needed, and value is either stable or enhanced.

The point which the writer desires to make is that ordinarily the work of the engineer in a rate case is not that of determining value, but rather a work of determining a basis for rate-making on which a rate of return may be predicated. The actual determination of value is the work of the public service commission in fixing the rate of return. The commission actually makes value. If the service rates are modified by a commission in a manner to change net earnings, the value of the property becomes changed proportionately. Value is a relative, not a definite, term, becoming in academic discussions a sort of "will-o'-the-wisp". Therefore, if engineers in the accomplishment of their work would leave out of consideration entirely the idea of "value", the confusion which arises in attempting to reason vaguely and academically from cost to value or from investment to value would largely disappear. The writer, therefore, would suggest that valuation work in a rate case be regarded as an effort to arrive at a basis for rate-making, or, to put the matter more simply, to determine a rate-making base as a foundation for the work of a public service commission in fixing a rate of return. The

engineer's work may be carried even a little farther than this, in an endeavor to throw light on what constitutes a proper replacement return to maintain the particular property which may be under consideration at the time at a 100% value. The other portion of net earnings, the interest return on the invested capital, is largely a matter of business, and is not to be determined purely as an engineering problem. Mr. Kiersted.

The principles which the author outlines as peculiar to the "Unlimited-Life" method of valuation are not new. They have been recognized on several occasions. The writer applied them in his testimony on the valuation of the water-works property of Los Angeles, Cal., in January, 1899, when the property was being appraised by a commission for purchase by the city.

In this case the writer ascertained the commercial value of the property by capitalizing the net earnings, and, in his discussion of the reasons for pursuing this method, stated, among other things, that:

"the income of a water-works property, after deducting the actual operating expenses, should afford a net income (earning) which should pay a proper interest upon the investment in plant and property establishment and in addition thereto should support a general maintenance fund for expenditures in betterments which will effectually perpetuate the life of the property. * * * It should be stated in this connection that the application of the maintenance fund should not be to displace alone the materials of construction which have wasted away through the agency of natural decay, but also to reinforce or displace those portions of the plant which may have been incapacitated by reason of plant expansion in response to a demand for an additional supply of water, or which may have become inadequate by reason of modern improvements, as is often the fate of machinery. * * * This method of valuation implies an indefinite continuance of earnings. But this inevitably must be the condition of a property which is a part and parcel of the established property of a progressive city, regardless of the question of vested title or the existence or non-existence of a franchise. For whenever the franchise rights shall cease by limitation and the water company operating under it no longer exists legally, still the property remains a living business, because a continuous operation of it cannot cease without injury to private and public interests. * * * Upon a transfer of possession after due compensation, the property remains intact as before the transfer, the net earnings remain an established fact, and continue to remain indefinitely susceptible to apportionment for the support of the investment and for the maintenance of the perpetuating fund. Therefore it is maintained that valuation by the capitalization of net earnings based upon equitable rates, when proper apportionment is made to perpetuate the plant, becomes proper. But the valuation of a property determined in this manner embraces a plant which is capable of rendering adequate service of the kind contracted for. Consequently, if proper expenditures have not been made progressively to maintain the efficiency of the plant, then at the time of appraisement such deduc-

Mr. Kiersted.

tion should be made, consistently, from the capitalized commercial value of the property as a whole, as will represent the necessary expenditures to produce a water-works plant of adequate capacity. In making this deduction the question of physical deterioration is subordinate to that of the useful life of the materials of construction. Though it is conceded that physical deterioration is in progress continually, still the rate, range, and extent of such deterioration cannot be definitely estimated, to say nothing of placing thereon a tangible value, nor can the limit of useful life be predicated upon such a basis. On the other hand, it must be conceded that the continuance, even of an inferior service, demands the exercise of due vigilance on the part of the management of the property to replace useless material, for, in a united system, no matter how complex it may be, a temporary disablement of any essential part of a system impairs the efficiency of the whole to a greater or less degree. Expenditures from the maintenance fund have removed such defects as may have been discovered in the past, and they undoubtedly can be made to eliminate them in the future. When the cost of (neglected) reinforcements and renewals to prolong or renew the useful life of the plant are estimated and deductions from a commercial valuation are made accordingly, then all has been done that can be done, consistently, in estimating the depreciating effect of deterioration."

The same principles, with regard to depreciation of the commercial value of a water-works property, is referred to more comprehensively in the writer's discussion* of his paper "Valuation of Water-Works Property" presented in April, 1897, in which the following statement is made:

"Should certain portions of the plant, however, be inefficient, the necessary cost to reinforce the deficient portions should be deducted from the commercial valuation derived from the net earnings; or, if reinforcement be impracticable, to replace it with such new portions of plant as will fulfill the requirements of an efficient service. * * *

"A rule of this kind should be comprehensive; therefore, if a water company has distinctly built for the future, then to the commercial valuation should be added the cost of the parts which safely and distinctly provide for this future; * * *."

Although there can be no question that the way of ascertaining the value to an investor of an unregulated public utility is by the capitalization of earnings, with due regard perhaps to the future, as well as to the particulars of deferred maintenance, under-construction, and over-construction at the time of valuation, as set forth in the foregoing quotations, still the Courts seem to have considered this method of valuation as not altogether conclusive. This method of ascertaining value may have suffered like others in its presentation; but, however this may be, such a method can have no significance in a rate case until the work of rate-making in all its particulars

* *Transactions*, Am. Soc. C. E., Vol. XXXVIII, pp. 208-209.

shall have been completed. Whenever the net earnings resulting from the rate-making processes become known, then, and only then, does it become a factor in ascertaining value. Mr. Kiersted.

In the ascertainment of a basis for rate-making, or of a rate-making base, the writer agrees with the author that there is much in the investment, unlimited-life, no-depreciation method deserving thorough consideration. Doubtless many of those interested in valuation work for rate-making, as are the engineers, and of those engaged in the work of rate-making itself, as are the public service commissioners, have given the matter consideration; but they are far from united in their views. The logic of those attempting to frame methods of valuation to suit the decision of the higher Courts is not free from serious criticism. Others, willing to accept the investment and unlimited-life theory, hesitate at the no-depreciation part of the theory, ostensibly on the ground that the Courts have not as yet approved of allowing no deductions for depreciation. The reason for this view of the Court may be due largely to the failure of appraisers in the past to confine themselves strictly to the investment theory, as well as to the manner in which these matters have been presented to the Court.

It is necessary, therefore, that the investment, unlimited-life, no-depreciation method be defined clearly if the hope is entertained of its acceptance by the Court. In this connection it is not clear that the term "investment" has been defined explicitly. If the view of a California banker, once stated to the writer, "once an investment always an investment", is to be accepted in its broadest sense, there may be grave doubts of the investment theory ever receiving recognition. It is necessary, therefore, to define investment. The author defines "investment or capital invested" as

"the aggregate of the reasonable and proper expenditures which have been made to render the property in question efficient for the purpose for which it is intended."

Investment is further referred to as follows:

"In regulating rates, consideration need be given only to the amount of capital reasonably and properly invested, without any deduction for depreciation.

* * * * * * * *

"It is to be expected, therefore, that in the case of many public service properties the owner will have had no opportunity to make investments the earnings of which will meet his replacement requirements. When there has been such a deficiency of earnings, the ordinary method of computing the annual replacement increment by sinking-fund, compound-interest methods, based on the original probable life or on the reduced value computed by any method and the remaining life, would be improper and inadequate. It is under such circumstances that recourse must be had to special methods of computing the depreciation or the annual replacement increment.

Mr. Kiersted. "In such cases as those just referred to it may be necessary to provide for a replacement at the full cost of various parts in the remaining years of life, or burdening the property with an allowance for the accrued deficiency of earnings, which is then to be amortized in a reasonable number of years.

* * * * * * * * *

"Under the Unlimited-Life Method there has been no return of capital. The appraisal is at 100% of the investment.

* * * * * * * * *

"When, however, the appraisal is made on the assumption of unlimited life and the owner is assured a fair interest return on the full amount of his investment besides replacement allowance as the same becomes necessary, the property will have a 100% bonding value, * * *."

The question arising is, what is meant by "investment" or "capital invested" in a specific case; for instance, a water-works property which has been carefully inventoried. The inventory is made to include simply the various items of property in use and useful to the operating utility, and to exclude all reference to the items of property which have been abandoned or replaced, for one cause or another, but which may be still carried on the books as a part of the capital investment. Is the author's view of the investment for rate-making purposes confined to the inventoried property, or does it embrace all the property carried in the capital account? It is presumed, if it refers only to the inventoried property, that the owner must have been compensated through a replacement fund for all the property which is abandoned or for any reason displaced; but, in the event that the earnings have not been sufficient in the past to amortize the original cost of the abandoned articles, it is presumed the author considers that the investment in such items of property should be carried in the capital account until amortized. Thus, the capital account would be composed of two distinct elements, one of unlimited life perpetuated from replacement earnings, the other of limited-life in process of amortization. As an alternative, assume that unlimited life applies to both elements in the capital account, without any provision for amortization; then the owner would be drawing interest continually on the abandoned property, and interest and replacement on living property. On the other hand, if all abandoned or displaced property is charged off the capital account, as it should be at the time of replacement, regardless of whether the owner has or has not been reimbursed, and in its place additions are made to the investment in the property inventoried in an amount representing the past deficits, then the situation is again complicated under the unlimited-life theory because of the fact that the deficits should remain in the capital account only until amortized. Again, there is the combination of limited-life and unlimited-life elements in the investment.

These inconsistencies may be eliminated by restricting the rate-making base to the investment in inventoried property with such additions for interest, contingencies, development cost, etc., as may be fair and proper, allowed once for all, leaving out of consideration altogether all questions relating to the amortization of the original cost of abandoned or displaced property, and considering past deficits to such extent only as may be open for consideration as a part of operating expense to be carried in the operating account until amortized in some limited period of time that may suit a particular case. This would apply to property that is being appraised for the first time for rate-making purposes by a public service commission. Thereafter replaced items of property should be charged off capital account, as replacements may be made, and the substitute article added thereto, with due allowance for scrap value—the rates being modified by the public service commission from time to time as occasion may require, but always predicated on the rate-making base, as at first determined, corrected by additions and subtractions in the manner above indicated, with due allowance for interest and other expense properly chargeable to piecemeal construction. Mr. Kiersted.

Whether deductions for depreciation shall or shall not be made in determining a rate-making base depends on the method pursued in making the appraisal. It can scarcely be conceived that a public service commission should confine itself to any particular method of valuation. Undoubtedly, a careful audit should be made of construction accounts with a view of determining as far as possible the cost of the property in use at the time of the appraisal. In a similar manner, an audit should be made of income, disbursements, and earnings. However, such an audit, under most circumstances, can scarcely be conclusive evidence of actual investment.

The records of the cost of construction of most of the smaller public utilities, and perhaps of some of the larger ones, can be verified by estimating the cost of construction made piecemeal by the use of unit prices made up to suit the price of materials and labor prevailing at the time each particular piece of work was done. This procedure, of course, necessitates a careful search of the construction records, and the tabulation of the construction work in a manner to show the original construction and the annual construction increments from year to year. In pursuing this method, considerable benefit is to be derived from a tabulation of the prices of materials and labor showing the variations from year to year. Doubtless a search will disclose the existence of much desirable information of the kind. A late investigation by the writer along this line is illustrated by two typical labor diagrams, Figs. 8 and 9. Fig. 8 represents the general percentage variation of the hourly wages paid to bricklayers from 1840 to 1913, and Fig. 9 represents the hourly wages paid to laborers for the same

Mr. Kiersted.

period. Each diagram presents a double curve during the Reconstruction period, after the Civil War; one curve, through a series of points (circles), shows the percentage variation of wages paid in currency, the other curve shows the equivalent variation for payments in gold. A similar set of diagrams for each class of labor and for manufactured materials, like cement, cast-iron water pipe, copper

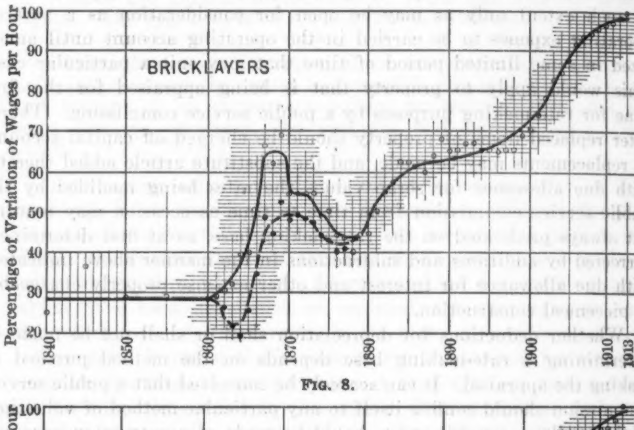

FIG. 8.

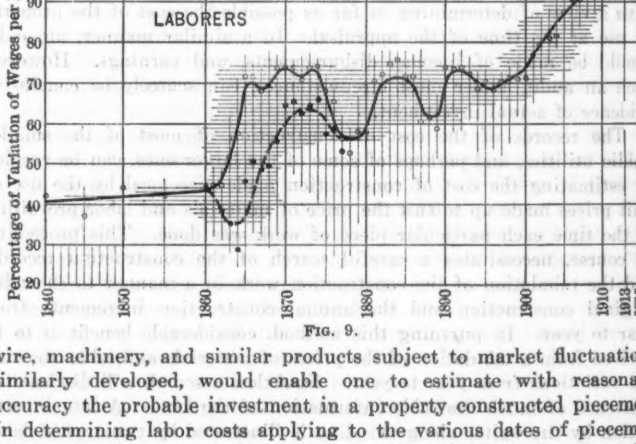

FIG. 9.

wire, machinery, and similar products subject to market fluctuations, similarly developed, would enable one to estimate with reasonable accuracy the probable investment in a property constructed piecemeal. In determining labor costs applying to the various dates of piecemeal construction for a particular locality, it will probably be found possible to trace out smoother and more regular curves than those shown on these diagrams.

Mr. Kiersted.

In the absence of records showing the piecemeal development of a property, the reproduction method of estimating cost would have to be pursued; but the use of that method should entail a modification of some, at least, of the present processes, if no deduction is to be made for depreciation in arriving at a rate-making base, as, for instance, a modification to exclude undue weight for those elements frequently referred to as the unearned increment, some of which, like the paving over pipe lines placed after the pipe lines themselves were constructed, have aroused strenuous objection. In this connection the author allows the present value of real estate by regarding the increased value thereof as reinvested earnings. Still another reason, referred to by the writer in several previous discussions, is that the present value of real estate should be allowed, on the presumption that the increase in value is due to urban surroundings, particularly in a growing city, rather than to its identification with a particular public utility. In other words, present value of real estate should be considered on the ground of its usefulness for purposes aside from that due to its particular association with a public utility, entitling it to different consideration than that accorded items of property possessing value that is due wholly to a specific use as part of an operating property.

Before passing this part of the discussion it may be pertinent to add a word with reference to an estimate of reproduction cost under original conditions at present-day prices of materials and labor and under present-day methods of construction. It is a compromise of the investment method suggested by the author and the method of reproduction under present-day conditions as ordinarily pursued, with a view undoubtedly of avoiding obvious difficulties peculiar in both methods, respectively, when logically pursued. It would seem, however, that the historical records and the book accounts affording explicit information as to past physical conditions would present with equal completeness the actual cost under the original conditions and would otherwise be sufficiently explicit as to dates, etc., to enable one to apply the prices of material and labor prevailing at the time of construction with sufficient accuracy to test the book account of costs.

With judicious discrimination of the kind indicated in estimating the probable piecemeal investment in a public utility, or the probable cost of reproduction as estimated by a restricted method, together with a comparison with actual cost records contained in the book accounts, and with due allowance for development expense, interest, and the necessary overheads, it would seem, with regard to many of the monopolistic utilities, that there should be no serious difficulty in arriving at a rate-making base without further allowance for depreciation than that introduced in a process of valuation which excludes consideration of investments made in property abandoned or which, for one cause or another, may have passed out of use. The deter-

Mr. Kiersted.

mination of a rate-making base is regarded by the writer as separate and distinct from the determination of the value of a property for purchase and sale.

Before concluding this discussion the writer desires to call attention to the importance of selecting proper unit prices in valuations for a rate-making base. Each unit price should be thoroughly investigated before use in valuation work, and the various steps taken in the make-up of the unit price for each important item of property should be presented analytically. Such procedure will tend to avoid the unbalanced estimates of physical property sometimes encountered in valuation proceedings, and will compel the engineer making the estimates to be thorough in his work and careful in his selections. With few exceptions, the more nearly this analysis conforms in its make-up with past physical conditions and past market prices of labor and material, the more nearly will the ends of justice be served in determining the amount of money actually and fairly invested.

In conclusion, the writer would summarize the points he has endeavored to bring out in this discussion, with due regard to the stated restrictions, as follows:

1.—The process of rate-making consists of two independent operations: one is the determination of a rate-making base, the other is the determination of a schedule of rates which will insure proper return on an investment previously ascertained.

2.—The value of a public utility rendering an efficient service is a function of net earnings.

3.—Valuation is the initial step in rate-making, and, so far as it goes and can go, is not in itself a method of determining value. Accordingly, all idea of value should be excluded from the process of valuation for rate-making. Valuation in a rate case should be directed to the determination of a rate-making base.

4.—Investment in property in use and useful for the rendering of an efficient service and in the judicious development of such property is a term synonymous with rate-making base.

5.—Investment thus defined may be determined in part from construction accounts, or from estimates of the cost of constructing and developing the property piecemeal, or from estimates of reproducing the property under substantially present-day conditions, when excluding undue consideration of the unearned increment. As a rule, the determination of a rate-making base should give due consideration to all three methods herein referred to.

6.—From a rate-making base determined in the manner above stated no deductions should be made for depreciation.

7.—Investments in public utilities, as a rule, should be regarded as of unlimited or perpetual life, and in addition to earning a fair rate of return, they should earn sufficient to replace items of property

as they become worn out or obsolete, and to provide reasonably for unforeseen contingencies. The obligation to replace such items of property is direct and imperative. Replacements thus made perpetuate the life of the property. Mr. Kiersted.

8.—Elements of cost open to consideration, but which should not remain permanently a part of the rate-making base, should be amortized as a part of operating expense during such term of years as it may be desirable to fix in any particular instance.

9.—Properties under regulation by public service commissions should have added to the rate-making base the net accretions made to the investment from time to time as the property extended and improved.

10.—Unit prices used in making up a rate-making base should be carefully selected, compiled, and itemized.

11.—The rate of return on the rate-making base is not always controlled by the hazards to which an investment is exposed, nor by the rate of return common to other similar investments, for the reason that social and industrial conditions must receive due consideration. That is to say, declining social and industrial conditions in a town or district must result in a decline of earnings and in a small rate of return on a fixed rate-making base.

J. N. DODD,* ESQ. (by letter).—The importance of this subject is well recognized, especially at the present time. Depreciation of properties, it is true, has always existed. Practically all properties become less valuable with age. Appraisals have been made since commerce began. Depreciation has always been considered as an element in such appraisals, in fixing the price which a purchaser should pay. However, not until lately have appraisals become of importance to the economic life of the community. As long as a business was subject to unlimited competition and the purchaser of a commodity was not compelled to buy at one place or from one firm, the amount at which the owner valued his business, or indeed for which he was able to sell it, was without influence on prices, and had very little effect on the community. Exchange of the property at too high or too low a figure was in no way a destruction of wealth. An attempt to capitalize the property at too high a figure and to compel the community to pay prices which would permit the prevailing rate of return on too high a valuation was fatal, as competitors were always ready to undersell. Competition, therefore, provided ample safeguards for the public. Mr. Dodd.

With the growth of modern public utility corporations, however, such safeguards no longer exist. These corporations deal with necessities, so that their products are those which the public must buy. Such corporations are virtually, if not actually, monopolies, and therefore the public must buy from them.

* New York City.

Mr. Dodd.

Moreover, it is provided by law that the corporation is permitted to earn a fair return on a fair valuation of the property. Therefore that valuation has an immediate and vital influence on the community, as it determines the amount which the community must pay for the commodity. For this reason, depreciation, and the extent to which it affects valuation, has become an important question during the last few years.

Definition of Depreciation.—As generally understood, depreciation may be defined as the loss of value of an article or property through age or through use. It includes, therefore, loss of value: through failure to give the original service; through increased cost of maintenance; on account of change in the demand; on account of the development of new and more economical machinery; and through the approaching date when the article or property must be replaced. All these causes affect vitally the value of the property.

The foregoing definition does not agree in all respects with the one given by Mr. Grunsky, who defines depreciation as:

"the lessening in worth of any perishable article or property. It is not, therefore, to be measured by inherent deterioration, due to wear and tear, but solely by a comparison between the probable life, the expectancy [the remaining life], and the cost of replacement * * *."

With the first part of the author's definition it is impossible to take exception. It is precisely the one always assumed in the appraisal of any property, whether owned privately or publicly, and whether the appraisal is for the purpose of purchase or of rate-making.

The second part of the definition, however, is open to criticism. It is difficult to understand how, with the first sentence as a premise, one can obtain the conclusion in the second sentence, as the author does.

Depreciation is a lessening in worth of a property, and it is due to several causes, not to one. An article may at any time show very little wear and accordingly have a very long remaining life. Yet, on account of changes in the methods of production, or in the demand for the product, the article is useless. The value of such an article, therefore, is lessened and the depreciation is high, but this lessened value is not on account of its short remaining life. The development of electricity for traction practically annihilated the value of the cable plants in New York City for the operation of street railroads, in spite of the fact that those plants had barely been installed, and, with ordinary usage, had many years of life ahead of them.

Necessity of Allowing for Depreciation.—If capital is to be invested in any business, two things must be assured, namely, security of the investment and a fair return on the capital.

Depreciation means a disappearance of the capital invested. Therefore, before capital can be secured for a project, there must be

assurance that the probable income shall be such as to permit, not only a fair return on the investment, but also full depreciation allowance. Mr. Dodd.

Methods of Recovering Depreciation from Earnings.—Depreciation may be recovered from earnings in two ways: An amount sufficient to cover the depreciation loss may be returned from earnings to the investor, thus reducing his investment by an equal amount; or this amount may be reinvested in the business, thus keeping the capital up to its original amount.

In case of a business owned privately, it matters little which method is used. If the business is one which depends on a whim of fashion, it is well for the owner to try to reduce his investment at the earliest possible date by taking from the business the depreciation earned. If he wishes to remain in the business or to leave it on a firm financial basis, he will keep up the capital by returning to the business an amount equal to the loss by depreciation.

In case of a private business, it matters little to the community which method is used in recovering depreciation from earnings; whether the capital be lessened or whether it be retained at its full value.

In case of a business publicly owned, such as a stock corporation, and especially in case of a public utility company, it is usually imperative that the capital be maintained at its full value. Practically, the only exception to this is the case of a mine or like property. The value of a mine is decreased by the ore taken from it, and eventually the mine is valueless and the invested capital disappears.

In general, however, the possible investor must be fully protected against investing in a company which, under the guise of paying large dividends, has actually been repaying to the owners amounts which should have been credited to depreciation. Many of the scandals in the financial world are due directly to such practices. If the business concerned is that of a public utility company, the necessity of holding the investment intact is even more essential. The community has given the company a monopoly of the right to supply a certain commodity. In return for this grant, one of the rights of the community is continuity of service. The author admits (page 749) that "the 'obligation to replace worn-out parts' goes with every public service property." However, the only way to guarantee that the owner will replace such parts is to forbid him to withdraw from the property, even temporarily, the depreciation allowance earned.

In this connection, there are two statements of the author with which it is impossible to agree. On page 746 is found the following:

"The amounts thus received [by the owner to cover depreciation] are usually held to be repayment of invested capital. This is a cor-

Mr. Dodd. rect view when their aggregate amount without interest will in the life of an article amount to the cost of that article."

Apparently, the conclusion from this statement is that, if the owner receives in this manner 100% of the value of the article, the full value must be written off the capital; but if he receives only 99% of the value of the article, the capital remains at its full original amount.

The author defines "depreciation" as a "lessening in the value". A corresponding amount, therefore, should be written off the capital simultaneously with the depreciation. It is impossible to reach any other conclusion than that the amounts received by the owner to cover depreciation are repayments of invested capital, whether he receives the entire amount or only a part.

Again, on page 759, he says that depreciation or amortization repaid to owners can be written off the invested capital only when one of three methods of calculating depreciation has been followed, and that the depreciation thus calculated has been earned. It follows from this that, if any owner should sell part of his plant by regular or irregular installments, or if for any other reason the value of the plant be decreased, the amount of such decrease cannot be written off the capital unless he also receives regularly in a given manner corresponding sums from earnings.

Manifestly, it is impossible to agree with this conclusion. Depreciation is real. Unless the loss is restored, it must be written off capital at once. The sums received by the owner to cover depreciation are repayments of capital, regardless of the law followed, and indeed regardless of whether any law is followed.

Various Methods of Computing Depreciation Allowance.—The author has described a number of ways of computing the correct depreciation allowance. Some of these, as defined by him, consist of a repayment from earnings of the investment, or as he terms it, "of an amortization of capital". Others are 100% methods in which there is no amortization of capital, but the investment is kept at its original value by the reinvestment from earnings of an amount computed to be equal to the depreciation loss.

As pointed out previously, there should be no repayment of capital in case of a public utility company. Accordingly, no method involving such repayment need be considered in connection with appraisal for rate-making. The author gives two methods of calculating the depreciation on a property and the amount of earnings that must be set aside each year for replacement in such a way as to keep the capital intact. One he calls the "Sinking-Fund Method". According to this, the depreciation allowance is the amount which must be set aside annually in a sinking fund at compound interest so that the fund,

at the end of the life of the property, will be equal to the cost of replacing the property. At first sight, this appears to be a very logical, if not the most logical, method. Mr. Dodd.

The second "100% method" suggested is what the author calls the "Unlimited-Life Method". According to this, the owner receives from earnings the annual depreciation allowance, but morally binds himself to replace the article at the expiration of its life. The amounts thus received by him must be set aside in a fund which must be used, principal and interest, for no other purpose than the replacement of the property at the end of its life. As the owner is not entitled to use this fund for any purpose, he is entitled to receive, from the business, interest on the original value of the investment.

Unfortunately, a better law is needed in business than the moral law. An absolute essential is that the sums thus set aside shall be under the control, not of the individual owner, but of the company itself. As pointed out before, no method should be permitted which allows, even temporarily, the withdrawal from the business of the depreciation allowance.

What is called the "Straight-Line Method" consists in setting aside from earnings an amount each year equal to the replacement cost divided by the probable life in years. According to the author, these sums are repaid to the owner, and this, therefore, is not a "100% method". As a matter of fact, however, it may be used as a "100% method", if these sums be retained in the company and re-invested either in the business or otherwise.

A criticism which has been made against the "Straight-Line Method" is that it requires the earnings to be as heavy during the early years as during the later ones, when the business is more fully developed and the payments can be more easily met. This may be a valid criticism. However, the depreciation is computed, not on the plant as a whole, but on the detailed parts, and usually, in case of an appraisal, most of these detailed parts actually included in the appraisal are provided after the plant is developed, and certainly the first years of the life of a machine are generally the most efficient and can best afford the heaviest depreciation charges.

In the Report of the Special Committee on Valuation, entitled "Valuation for the Purpose of Rate-Making", a method of computing depreciation was suggested which was designed with a special view to the criticism of the "Straight-Line Method" just quoted, and provided for increasing payments with age. Such a method is open to three criticisms: It provides for the heaviest payments during the latest and most inefficient years of the life of the property. It adds tremendously to the accounting difficulties. Imagine, for example, the labor of keeping the depreciation accounts on, say, 100 000 telephone poles divided by age into groups, no two of which are numerically

Mr. Dodd. equal, the original cost of the poles varying widely and the depreciation rate on each group differing from the rate on any other group. In this respect, either the "Straight-Line Method" or the "Sinking-Fund Method" has the advantage of great simplicity, as each provides that the annual depreciation allowance is merely a certain percentage of the sum of the cost of the replacement of all the poles. The method proposed is practically impossible of application except for a few units. Such units would naturally be large ones.

The third objection to this method is that it is not one of approximating the actual depreciation at any time, but only a formula for building up a fund which will probably equal the depreciation at the end of a certain period.

It is true that this last criticism applies also with equal force to the preceding methods of calculating depreciation. However, it serves as a strong argument against a method which is much more difficult than any other and, at the same time, is no more accurate.

Indefiniteness of the Problem of Determining Depreciation.—Depreciation of a property is real. Almost every item in the inventory is subject to it on account of one or more of the causes given in the definition, and the total depreciation may, and often does, attain a very high proportion of the original value of the property.

Nevertheless, methods of calculating depreciation are essentially indefinite, and must be suited to the conditions of the case. The life of an article varies, of course, with the usage it receives, with the demands on it, and with the character of the maintenance; but, even under the same care and the same conditions of service, the life of two pieces of apparatus apparently identical in construction and material will differ greatly from each other. In other words, actual depreciation cannot be predicted beforehand.

More than this, it is impossible to determine the actual depreciation which has occurred. It is seldom that one can find two people to agree on the value of an article: as measured by its remaining life; as measured by the service it can render; as measured by its increased maintenance charge; or, in fact, as measured by any of the elements that affect the depreciation. This being so, it follows that no hard-and-fast rule can be observed in calculating depreciation. When accounts are begun, a method should be adopted which one has reason to believe will be approximately correct. The method, however, and the amounts periodically set aside must be changed from time to time as the condition of the property dictates. It is impossible, therefore, to agree with the author when, on page 766, he insists that once a method has been adopted, it must never be changed.

Typical Method of Allowing for Depreciation.—In order to get a clear idea of the principles underlying valuations, it is advisable to consider briefly the accounting method followed by a typical firm,

conservatively managed, which tries to keep its capital up to the original value. A depreciation fund is built up by crediting to it from earnings amounts equal to the estimated losses in value of the plant during the year. This loss in value is the loss as considered from all points of view, not merely the loss as measured by its life, or service value, or any other of the single elements going to make up its value. Mr. Dodd.

If untouched, this depreciation fund plus the remaining value of the plant should be equal to the original investment. However, the fund is not usually untouched. It matters not to the investment whether the fund is allowed to accumulate, whether it is invested, or whether it is spent for betterments or for extensions. In any case, the value of the plant is: The depreciated value of the original plant, plus the amount in the depreciation fund, plus the depreciated value of the extensions and betterments. If the methods of estimating are correct, the total value at any time is not less than the original investment.

Most companies are constantly extending their property, and the amount in the depreciation fund is usually spent for extensions. When the items must be replaced at the end of their life and there is not sufficient in the depreciation fund, the amount may be obtained by a bond issue or increased stock based on the additions to plant.

When accounts are kept in such a manner, it becomes difficult to credit any interest to the depreciation fund. The fund being reinvested in the plant, the interest is treated as earnings. The amount credited to the depreciation fund varies from year to year as the condition of the plant dictates. From this may be seen the value of the "Straight-Line Method". It is simple. It is at least as accurate as any other, and adjustments can easily be made from time to time as conditions dictate.

Theories of Valuation.—Many of the author's statements, to which exception has been taken, are, perhaps, based on a certain theory of valuation which he gives. On page 758, the statement is made:

"There can be no fairer nor better way of determining the earnings of a public service property than that under which the owner is regarded as the agent of the public".

On page 757, are the words:

"When an appraisal is to be made, for rate-fixing purposes, of a plant which has continuously been operated with inadequate earnings, there is no escape from the conclusion that the investment has remained undiminished".

Difficult as this latter statement is to accept, the truth of it must be admitted if the first statement is granted. If the owner is an agent and erects the plant purely for the service of the community and as the agent of the community, clearly he is at all times entitled to a

Mr. Dodd. return on the original investment, whatever may be the condition of the plant. Such a theory has been strongly presented by others. However, the fact remains that the community does not engage the owner to erect and operate the public utility plant. Such a theory brings the certain conclusion that if an owner makes a mistake and erects a plant not suited to the needs of the community, the community is nevertheless compelled to pay him full interest on his investment. Desirable as such conditions might be to the investor, it breaks every economic law; it nullifies the exercise of sound business judgment; it discourages the ingenuity of manufacturers in developing better types of machinery, and penalizes the community.

The rules governing the installation and operation of a public utility company should differ very little from those governing any other company. The community gives the company a monopoly in a certain trade. This should produce a certain income. The owner of the property has the opportunity of judging what this income should be and of erecting a plant accordingly. He has no right to expect the city to pay for his mistakes.

On the other hand, the city has the right to good service, to all the improvements in the art, and at prices no higher than if it is protected by competition. In other words, depreciation due to any and every cause should be taken account of in the valuation just as mercilessly as if competition were in full force. No other plan is economically correct.

Conclusions.—Depreciation is the lessening of the value of a property for any cause, whether due to decreased efficiency of the plant, to development of improved machinery, to change in demand, or to the approaching date of retirement.

This loss is real, and, unless replaced, must be written off the capitalization.

The amount of such loss is not susceptible of exact statement or calculation. It cannot be foretold with correctness, and, when it has occurred, its amount at best can be only approximated.

The owner of a property has a right to expect earnings sufficient to cover the loss of capital due to depreciation, in addition to a fair interest on the investment.

In a public utility company, the amount so earned to cover depreciation loss must be reinvested in the business.

For all practical purposes, the "Straight-Line Method" is perhaps the best for calculating depreciation. However, it must be used with the understanding that it is only an approximation, and the total amount must be adjusted from time to time as the condition of the plant demands.

Mr. Bosley.

WILLIAM B. BOSLEY,* ESQ. (by letter).—The writer has read this paper with much interest and considers it a very valuable contribution to the literature on this subject.

Mr. Grunsky has done well to emphasize the fact that any method of providing for depreciation of public utility property must be continuously and consistently applied, from the beginning of operation, if results fair both to the owner of such property and to the rate-payers are to be attained. He might well have added that no method can be applied with even an approximation to mathematical accuracy unless the public which regulates the rates also guarantees the integrity of the capital properly invested by the owner, as well as regular earnings sufficient in amount to cover the cost of operation and maintenance, to make provision for all necessary replacements, and to afford a reasonable return on the capital invested and reasonable compensation for the service rendered by the owner. In the case of privately owned public utilities, however, the public, which through divers governmental agencies exercises the power to regulate rates in the interest of the rate-payers, assumes no responsibility whatever for preserving either the integrity of the owner's capital or the sufficiency of the earnings, and, consequently, the problem which presents itself to legislative or administrative bodies vested with the power to regulate rates and to the Courts when called on to review the action of the rate-fixing body, is how can they best apply the constitutional rule that private property may not be taken for public use without just compensation or due process of law.

It is settled by judicial decisions construing and applying the Constitution of the United States that the owner of public utility property cannot be compelled to give to the public either the corpus of such property or the use thereof without just compensation, and that just compensation for the use of such property, if subject to deterioration because of such use, includes a reasonable allowance for the depreciation in value resulting from such deterioration; but what constitutes a reasonable allowance for depreciation in any given case is a question of fact to be determined from evidence in accordance with principles of law. In the application of the principles of law to the facts concerning depreciation, there is still much uncertainty and conflict in the decisions of the Courts.

The Unlimited-Life Method, so ably presented by Mr. Grunsky, seems to the writer, not only to possess the great advantage of simplicity in both theory and practice, but also to be more capable than other methods of practical adaptation to the financial requirements of the owners of extensive public utility properties consisting of many units of different ages and not operated under franchises or rights of prescribed duration.

* San Francisco, Cal.

Mr. Bosley. The writer, however, thinks that Mr. Grunsky's statement that the Unlimited-Life Method is fair both to the owner and to the rate-payer should be qualified. It really is somewhat more than fair to the rate-payer and somewhat less than fair to the owner, unless accompanied by a guaranty of the integrity of the capital properly invested and also a guaranty of the sufficiency of earnings. For, even though this method be applied continuously and consistently from the beginning, the owner of public utility property subject to depreciation and not subject to appreciation, which has been in operation for any period of time, will be unable to realize, on a sale or condemnation of his property, the full amount of his investment, because there will then be some accrued depreciation or accrued "obligation to replace" which will be taken into consideration by the purchaser in determining what price he will pay, even though he confidently expects that rates in the future will be fixed so as to afford a reasonable return on the amount of the original investment and, in addition thereto, an amount sufficient to provide for current replacements. Nobody will pay as much for a fixed annual income, even if guaranteed in perpetuity, as for property which he is assured will yield a fixed annual income of the same amount, notwithstanding the fact that, according to the higher mathematics, any finite principal amount divided by infinity gives zero for a quotient. Moreover, it is contrary to all human experience that any public utility plant or property will continue in use for so long a period of time as to make the integrity of the capital invested a negligible consideration. For these reasons, as well as because the public does not guarantee either the sufficiency or the continuity of earnings or the integrity of capital invested in privately owned public utility plants and properties, the owner of a public utility property should be permitted to amortize, through appropriation of earnings whenever sufficient for that purpose, the amount of all accrued depreciation in excess of the cost of replacements actually made.

In view of the fact that the public does not undertake to indemnify the owner of a public utility plant or property against loss arising from its operation in the service of the public, it should not concern itself with the profit which the owner may make as the result of his intelligence, industry, or good fortune, provided only that the rates charged by him for commodities furnished and services rendered are reasonable. Reasonable rates cannot, in the nature of things, exceed what consumers can afford to pay, nor, on the other hand, except for short periods of time and under exceptional circumstances, be less than the cost of production, including a reasonable allowance for depreciation, a reasonable return on invested capital, and reasonable compensation for the service rendered by the owner in producing, distributing, and delivering commodities or rendering service to the public.

In determining what are reasonable rates, accrued depreciation as distinguished from accruing depreciation ought not to be considered, because, in the first place, if the public utility plant be maintained in a proper state of efficiency, the value of the commodities furnished or services rendered by means thereof is not affected by the accrued depreciation; and, in the second place, if the Government were to undertake to serve the public and to construct a new plant for that purpose, it would have to base its rates on the cost of such new plant. Current or accruing depreciation, on the other hand, is vitally important in every attempt to determine the reasonableness of rates, and at best the amount thereof can be estimated approximately on the basis of general engineering experience in much the same manner as insurance premiums are determined. Mr. Bosley.

C. E. GRUNSKY,* M. AM. SOC. C. E. (by letter).—The writer desires to express his appreciation of the reception accorded this paper, and is pleased with the interest manifested in his attempt to secure the recognition of correct fundamental principles, even though some of these appear to be at variance with the opinions of the Courts. It is gratifying to hear on many sides that the principle for which the writer mainly contends—the elimination of accrued depreciation from consideration in making an appraisal for rate-fixing purposes—has long been desired by careful students of the question, and that only the barrier, interposed by the tendency of the Courts and of public service commissions to insist on the use of depreciated value as an element in the calculation, interferes with a more general recognition of this fundamental principle. Mr. Grunsky.

The Courts have said that "fair value" shall be the basis of the calculation, and engineers and economists have not been able as yet to make it clear to the Courts that some of their rulings need modification.

In the matter of value, the Supreme Court of the United States has said:

"But the value of the property results from the use to which it is put, and varies with the profitableness of that use, present and prospective, actual and anticipated. There is no pecuniary value outside of that which results from such use." (154 U. S. Sup. Ct., 445.)

In the same case, the Court says:

"The basis of calculation is the 'fair value of the property' used for the convenience of the public. * * * Or, as it was put in the San Diego Land and Town Company *vs.* National City case * * * 'What the company is entitled to demand, in order that it may have just compensation, is a fair return upon the reasonable value of the property at the time it is being used for the public'."

* San Francisco, Cal.

Mr. Grunsky.

This is logical only if a forced interpretation is placed on the expression "fair value", or "reasonable value", otherwise the reasoning will be in a circle. Pecuniary value results from the earnings, and value, therefore, cannot be made, as the Court would have it, "the basis of calculation".

The general rule should be to establish rates which will give the public utility property a "basic value" which will bear a fair relation to the legitimate and proper cost of the property. This should be done with due regard, not only to the present and prospective demands for service, but also to the past history of the utility.

The resulting "basic value" thus established is not the amount which a purchaser would pay for a public utility plant, but it will be his starting point in determining how much he can afford to pay. He will deduct therefrom the accrued obligation to replace and the deferred maintenance, when he makes an estimate of the present worth.

The "basic value" which results from the permitted earnings, whether immediate or prospective, is not to be confounded with the "rate base". It should bear a proper relation to it, however, and to the investment that would have to be made to re-establish the plant new, due account being taken of public contributions toward the cost of the plant, which, in some cases, may have to be regarded as representing an interest of the public in the plant. It can be defined as the legitimate investment increased by the reward to which the owner is entitled for establishing and managing the enterprise and by the strategic value, if there is any in such properties as water rights and the like.

When the owner of the public utility is protected, as he would be under the recognition of the fundamental principle previously set forth, then appreciation may be disregarded in fixing rates, just as accrued depreciation may be disregarded. The appraiser will be concerned primarily in ascertaining the proper and legitimate cost (investment) instead of the "fair value", as heretofore.

There will be cases in which the demand for the service rendered by the utility is so small that no reasonable rate will yield sufficient revenue, present or anticipated, to give the property a value anywhere near the original investment. In this event, the enterprise is a losing venture. Enterprises in which such conditions can be foreseen should not be undertaken except with financial aid from the community which is to be served.

There will be other cases in which, by reason of exceptionally favorable conditions, reasonable rates will produce relatively large earnings and establish values which may be far in excess of the cost. In this event, the owner's profit is large—perhaps unduly so. Nevertheless, it may be legitimate, even though based on customs, laws, or privileges which are unwise.

To illustrate, the case of a city may be taken the inhabitants of which are supplied with water by two concerns, A and B. Of these, one obtains water of good quality, but limited in quantity, from a lake near at hand at a sufficient elevation to make delivery by gravity; the other obtains water, requiring filtration, from a remote source, and must resort to pumping to effect a satisfactory delivery. Let it be assumed that the water of the two concerns as delivered is practically identical in quality, that neither concern has any material advantage in the matter of reliability of service, and that the consumers supplied by each are distributed indiscriminately throughout the city. Mr. Grunsky.

The rates established in this case should apply to both concerns. There is no apparent reason why those who happen to be so fortunate as to be served with water by A should pay less for it than those who obtain it from B.

It may cost A only 5 cents per 1 000 gal. to deliver the water, as compared with a cost of 15 cents to B. In order to hold his customers and to be sure of a market for his whole supply, A may charge a few cents less per 1 000 gal. than B. In such a case as this, the values of such intangible elements as water rights, going concern, and the like, of the property owned by A, determined from an income of about 10 cents per 1 000 gal. more than goes to B, will be large.

This is a case very similar to that of the riparian owner of a water-power which is convenient to an ample market. Here, too, if the development cost is low, there may be a large excess of value over cost, which will naturally be assigned to intangible elements appropriately classified according to the whim of the appraiser.

It is obvious that these values in excess of the investment are the result of the growth of communities, and that the large benefit to the fortunate owner of such a property is one of accident rather than of intention. The advantage that comes to a private owner from community growth is one that should in some measure be shared with the public, and there is good ground for believing that in the case of public utilities there should be some sort of partnership arrangement between the owner and the public.

This is put forth as an academic statement. It sounds good; but in the practical application of such a principle there may be some difficulty. If there is participation, there should be some voice in the direction of the business. If the original investment is to be subject to approval step by step, by properly constituted authority, there will be less incentive for private capital to embark on new enterprises than in the past, and the development of the latent resources of the country will be slower than would otherwise be expected.

The manner in which Mr. McCarty has stated some of the fundamental principles contended for by the writer is duly appreciated, particularly his clear statement with reference to what would justify

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a deduction of accrued depreciation from the legitimate investment. He points out that in fairness it can only be deducted when it is actually returned to the security holders, not as dividends, but as a return of capital.

The writer believes in the protection of legitimate investment in public utilities. He believes also in the protection of the rate-payer, and he is not, therefore, in full sympathy with those who claim that all appreciation of values should be included in the rate base. Such accessions of value are the result of community growth, and do not represent investment. This principle contended for by Mr. McCarty, as laid down in the Consolidated Gas Company case, should not be generally adopted.

In relation to the methods that should be adopted in ascertaining what amount should be regarded as properly and legitimately invested and to be considered in fixing rates, it need only be stated that methods, such as estimates of cost of reproduction, are to be regarded as means of approximation, and that no definite rule of procedure, applicable in all cases, can be laid down.

In his interesting contribution to this discussion, Mr. Henry calls attention to the attitude of the Internal Revenue Bureau toward the treatment of depreciation. According to this Bureau:

"Where a depreciation reserve is set up, all renewals and replacements must be charged to such reserve, and the addition to this reserve each year must be a fair measure of the loss which the corporation sustains by reason of the depreciation of its property."

If this regulation has been complied with from the beginning of operations, and the amount going into the reserve is used, as prescribed, to make all renewals and replacements, the amount that should annually go into the reserve should be estimated, not by any rule for determining lessening of worth, but by the rules laid down in the paper for determining the annual replacement requirement.

The writer has endeavored to make plain that, in so far as the fixing of rates is concerned, it makes no difference what the owner does with the replacement fund. It should be adequately protected, to insure the accomplishment of its purpose; but even when the owner fails to hold the fund intact, "the obligation to replace" remains, and the service rendered, if this obligation is fully met, does not suffer. Consequently, even though the replacement fund has been disbursed and this disbursement is construed as a withdrawal of capital, as Mr. Mortimer suggests, and as is frequently assumed, this should have no effect on the value of the service, nor on the earnings, and is no reason that the rate base should be otherwise determined than as contended for in the paper.* The taking of rate cases into the Federal

* See also "The Appraisal of Public Service Properties as a Basis for the Regulation of Rates", *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 828.

Courts is justified, as Mr. Mortimer says, because the confiscation of property is involved. This confiscation is involved just as much when rates are established which create an inadequate value, as when condemnation of the property is in question. Mr. Grunsky.

The use by Mr. Kiersted of the term "rate-making base" is approved, and has been adopted by the writer in this discussion with a simplification to "rate base". Mr. Kiersted is in accord with the writer's views when he contends that the appraisal for rate-fixing purposes should not be an appraisal of value, but a determination of the "rate-making base". He agrees with the writer that value in its ordinary sense cannot be made the basis for estimating the allowable earnings, and that, therefore, the appraisal of the "fair value" sought by the Courts as the starting point for fixing rates is out of place.

Despite the Court decisions, the "fair value", no matter how determined—"the cost new less depreciation" increased by "going concern"; "cost of establishing business"; "franchise"; or any other intangible elements of value, even though carefully evaluated—is not the logical starting point. The one item of prime importance is the actual legitimate investment. This is of fundamental interest to the rate-fixing or rate-regulating body. It does not follow that this investment, even when it can be determined with precision from the account books, or in some other way, is to be accepted without modification as the "rate base" in any particular case. It does, however, always become an important element for consideration. Neither can any hard and fast rule be laid down for its determination. Where the property has been well managed, where there have been no unnecessary employees, and salaries have been reasonable, the book accounts are generally the best guide; but where much unproductive or useless work has been done, or where there has been a large payment to promoters; where excessive salaries have been paid, or, for other reasons, the overhead expense has been out of reason, methods, such as the determination of cost of reconstruction and the like, may become necessary to approximate the legitimate investment which, in substantial agreement with the suggestion made by Mr. Kiersted, has been called the "rate base". Moreover, this rate base should come into consideration, as pointed out in the paper, regardless of accrued depreciation and deferred maintenance. This does not mean that the replacement requirements are to be neglected in estimating the rates which will produce adequate earnings.

It sometimes happens that the owner of a public utility receives assistance in the shape of a bonus. Perhaps conditions are not such that earnings can be hoped for, which, from a purely business standpoint, would justify undertaking the enterprise. The community to be served determines to extend financial aid, as in the case of the town which promises a high hydrant rate to a private company for a

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an article will probably go out of use is to be taken into account. The accrued depreciation is the difference between the original cost and the remaining value, and it is this last element which is to be estimated from the expectancy, the remaining years of service, and the cost of replacement.

A wrong conclusion has been drawn by Mr. Dodd from the statement made by the writer that allowances for depreciation are usually held to be repayment of capital. It is agreed that they may be so construed. If they are not computed so that without interest they will amount to the investment, then they will either fall short of, or exceed, the desired amount. Mr. Dodd, however, is not correct when he assumes that, because allowances are made for what is termed depreciation and what equally well might be called the replacement requirement, a corresponding amount must be written off the capital, because these amounts may instead be held inviolate to fulfill their purpose, to make replacements.

The statement by Mr. Dodd in his conclusions that the owner of a property has a right to expect earnings sufficient to cover the loss of capital due to depreciation in addition to a fair interest on the investment, is to be weighed against the writer's contention, that the owner need have no part of his invested capital returned to him until the purchase of the property is to be effected, but he should be allowed earnings sufficient to cover anticipated replacement requirements in addition to a fair interest on the investment, and should have adequate assurance that sooner or later he will recover his entire investment. The earnings, by the way, should be such that the owner will in time be suitably rewarded for establishing and maintaining the service.

The interpretation placed by Mr. Mayer on the language in the paper relating to remaining value was not intended, and is hardly justified when considered with the context. His computation of the remaining value of an article which when new had a probable life of 60 years and is estimated to have an expectancy of 20 years, interest at 6%, is correct, and agrees with what is set forth in the paper. When the article, the probable life of which new is 60 years, has attained the age of 66 years and is still in fair condition, its expectancy may be about 20 years, as indicated by some such curve as that in Fig. 6. Its remaining value (on the assumption that the replacement cost will be the same as the original cost) is found in the 60-year, 6%, amortization table, for the expectancy or remaining life of 20 years, to be about 71%, as shown, too, by Mr. Mayer's computation. This illustrates one of the important facts to which attention has been called in the paper. The remaining value is not to be determined from the age, 66 years, but from the expectancy of the article at that age.

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Mr. Mayer has interpreted the writer's definition to show a value of 33⅓% in the special illustration cited by him, but this would only be possible, under the definitions given in the paper, when the interest rate is zero, when the Straight-Line Method is used.

The comments of Mr. Bosley, an attorney of large experience in valuation matters, are particularly appreciated, and deserve attention. A vital question involved in the Unlimited-Life Method, as in other methods of procedure, is whether it is being correctly applied. The replacement requirement—the current amount to be allowed for replacements in the earnings—may be computed, under this method, for individualized articles, by recourse to annuities at compound interest, in the same way as current depreciation, in which event the distinction between this and the Sinking-Fund Method disappears. In estimating what the earnings should be, it is always necessary to follow a definite plan and to let this plan be determined by all circumstances, the past history as well as the anticipated business of the future. Mr. Bosley is on the same platform with the writer when he contends that, "except for short periods of time and under exceptional circumstances" reasonable rates cannot "be less than the cost of production, including a reasonable allowance for depreciation [or preferably for the anticipated replacement], a reasonable return on the invested capital, and reasonable compensation for the service rendered by the owner in producing, distributing, and delivering commodities or rendering service to the public". According to his views, however, if correctly understood, the plan of treatment should provide for the allowance of earnings, when not too burdensome to the rate-payer, that will be sufficient to amortize that portion of the capital measured by the accrued depreciation. The point which is made in the paper is that although this may be done, it is not necessary, particularly not when dealing with extensive complex properties; and certainly the owner of a public utility which has been in operation for some years will be better off and in a better position to sell to a purchaser if he is allowed interest on 100% of investment and a proper replacement requirement, than if he is allowed interest on only the depreciated value (say, 75% of his investment) and current depreciation.

The writer does not understand Mr. Bosley to mean that, if applied as intended, there is anything less than fair to the owner in the Unlimited-Life Method. His comments are directed to the application of this method and not to the principle involved.

In the case of a complex property made up of a multitude of items, the owner, under the Unlimited-Life Method, if advantage has been taken of the low replacement requirements of early years, may be allowed somewhat less earnings in these early years of operation than will offset from year to year the lessening of worth when estimated from a consideration of the remaining value of each item;

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but he will be fully compensated for making each replacement. He will have made an investment of 100%, on which he is continuously allowed interest. He will not be required to add to this investment as renewals become necessary, because these are fully provided for out of the earnings. He will be held to accountability for the portion of the earnings which are allowed to him for making replacements. When the earnings of the early years do not permit amortization of accrued depreciation, this accrued depreciation remains a part of the investment, and should be allowed a fair interest return, and, if it be thus allowed to earn a return, it has value (Unlimited-Life Method). The owner will be better off, if this fact is recognized, than he would be if he were allowed to earn return on the depreciated value, and had to depend on "going concern" and other intangible elements of value to cover the unrecovered accrued depreciation.

If, on the other hand, a current depreciation allowance can be and has been earned from the beginning, then an amount equal to the accrued depreciation will have been returned to the owner as a part amortization of the invested capital. This is just as true as though a lump-sum cash contribution of the same amount had been made to him by the rate-payers. He will be entitled to interest on the remaining investment as well as to the current replacement requirements.

If a plant is under consideration, which is made up of numerous parts and has been in operation for many years, the current annual replacement requirement will be about the amount computed by the so-called "Straight-Line Method". This replacement requirement, if actually earned, together with interest on 100% of the investment (Unlimited-Life Method), will be somewhat greater than the amount which would be allowed under the "Equal-Annual-Payment Method". This is true, notwithstanding the theoretical fairness of both methods, if consistently applied from the beginning.

When the earned annual depreciation increments go into a fund which is separately accounted for, this fund—except for the irregularities resulting from non-conformity of actual with probable life—will supplement the remaining value. Similarly, if the replacement requirement for a single or individualized article is estimated in the same way as depreciation is customarily estimated (cost and replacement cost for purposes of illustration being taken to be the same) and is actually earned from the beginning of operations, then there should be an accumulation in the replacement fund equal to accrued depreciation. A purchaser will, if this fund becomes his, pay 100% for the property. He will not pay 100% if this fund can be or has been withdrawn by the former owner, because he will have to make it good. The interest on this fund is necessary to make up the current replacement requirement. Such a fund is theoretically not necessary under the

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Unlimited-Life Method if, instead of the current depreciation with interest thereon, the actual full amount of the replacement is allowed as needed.

In the case of the ties of a railroad the accrued depreciation may be 40 per cent. The owner may not have recovered this accrued depreciation. What will a prudent purchaser pay for these ties? He is allowed the cost of a new tie (full value) for every tie that goes out of use. He is allowed interest on 100% of its cost. In the course of about 10 years, all ties will have been renewed. He will have recovered and have re-invested in this time 100%, and will be still in the same position as the original owner found himself when he made the sale. If he is allowed interest on 100%, and is sure of a continuance of this allowance, the value is 100%, and he can afford to pay this price.

The comments of Mr. Bosley, as already stated, should be considered, therefore, as addressed to the application of the Unlimited-Life Method which, like any other, may be applied so as to be unjust to one party or the other. If properly applied, it is as fair to the owner as to the rate-payer, and it will admit as well as any other method the making of suitable allowances in the earnings for the amortization of early losses or of any other items which should not be carried indefinitely in the rate base nor as an addition thereto.

If properly kept, the records of expenditures should be the best source of information in determining the rate base. An estimate of the cost of reproduction may be a valuable aid in making an approximation of the investment. The cost of establishing the business will require special investigation, to determine the fair allowance that should be made therefor. The inclusion of other elements, such as water rights, in the rate base is a debatable one, even in the case of the value which results from strategic position and environment, (except, of course, to the extent of actual legitimate cost), because the water right and the franchise, being grants by the public to the owner, should have only such values as are deliberately sanctioned and as are created when pre-determined earnings in excess of interest on investment are permitted. When such elements are excluded from the rate base, however, then the earnings should yield more than the fair rate of return on ordinary business ventures, and a capitalization of these excess earnings should represent the reward to which the owner is entitled covering all values of intangible elements not included in the rate base.

In the special case of communities of an evanescent character, existing only pending the accomplishment of some one object, as in the case of the town which dies with the exhaustion of a mine, or with the completion of a great engineering work such as a dam or a canal, there may be public utility properties, the probable life of which is

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limited. In such cases amortization of the capital must be included in the earnings, and, if the probable life of the utility is short, this amortization sometimes takes the place of the replacement requirement.

The application of methods of procedure which involve consideration of depreciated or remaining value as a part of the rate base is fraught with difficulty, and, unless the application is consistent from the beginning, injustice may be done to either the rate-payer or the owner. This has led to recourse to special allowances of value for "franchise" and "going concern", "the cost of establishing the business", and "appreciation". Of these methods, that which involves the determination of the lessening of worth by the "Straight-Line Method" is the crudest, but by reason of the ease with which the computations can be made, it has become popular.

The Unlimited-Life Method stands apart. Its advantages have been pointed out. It can be applied without estimating accrued depreciation, and it permits, when this appears desirable, relatively low earnings in the early years. In the application of the Unlimited-Life Method, however, there should be the same assurance, as in the case of any other procedure, that it will be consistently adhered to. Under this method, as under any other, the owner must be in a position to realize on his investment without loss whenever he desires to do so. He must be in a position to transfer his interest and to assure his successor that a 100% valuation will be continued, and that ultimately the capital will be paid back at the termination of a franchise or at some indeterminate period.

In other words the public must assume the inviolable obligation to protect the investment so that well-advised public service enterprises can be conducted without fear of sacrificing any part of the investment.

When rates are to be fixed:

There is no need of ascertaining present value and, therefore, no need of estimating accrued depreciation.

The "rate base", the amount of capital legitimately invested, is a proper and essential element for consideration.

Losses incurred during the period in which business was being established, deferred maintenance, and generally the past history of a public utility, are elements which should be given due weight.

The anticipated business must be taken into account, particularly in the case of a new enterprise, where the rate-payers may be too few at the outset to produce, at a reasonable charge for the service, the revenue which will yield a proper return on the investment.

The immediate and prospective replacement requirements must be ascertained, with a view to making the earnings adequate to meet the same, and these requirements must be assumed to mature at the time the articles go out of use.

No provision, other than covered in that for replacement, need be made for the amortization of capital, except in case the earnings are to include a purchase increment or its equivalent. Mr. Grunsky.

Intangible elements, such as franchise and going concern, the value of which results from the surplus of earnings over and above interest on the rate base, are not a part of the rate base as here defined. Their value is to be created as a reward to the owner of the utility and in this sense only do they come into consideration.

The replacement requirement is to be determined from the expectancy (remaining years of usefulness) and the estimated cost of effecting the replacement. It is not dependent on original cost nor age, although original cost and age may frequently be valuable aids in making an estimate thereof.

Appreciation belongs among the earnings, and is not, therefore, to be made a part of the rate base.

The rate base should preferably include only a few elements besides the capital invested, perhaps only a reasonable allowance for the cost of establishing the business.

Suitable provision must be made to amortize a proper allowance for establishing the business and early losses when these elements are legitimate and have not been included in the rate base as a part of the investment.

By an oversight, the writer, in the sentence commencing in the seventh line on page 732, allowed himself to state that "fair value" is based on proper and reasonable investment, which may be ascertained from cost records—"and when cost records are not available, is usually estimated by the 'cost-of-reproduction' method". The quoted clause should have been made to read: "and when cost records are not available, the investment is usually estimated by the 'cost-of-reproduction' method". It may also be added that the context shows that the "fair value" relates to value for rate-fixing purposes.

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THE VALUATION OF PUBLIC UTILITY PROPERTY*

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With Discussion by Messrs. Alex Dow, F. Lavis, Charles Rufus Harte, F. W. Green, Joseph Mayer, and J. H. Gandolfo.

Synopsis.

The primary object of this paper is to show that, in any valuation or appraisal that is to be made for purposes of rate-making or security issues, the proper figure to be used is the actual cost to date of the work in question; provided, of course, that there has not been extravagance, fraud, or gross mismanagement in the origin or development of the enterprise. At the same time, it is also shown that, although this value is a basic figure, it is not necessarily the only and final one to be used for any and every purpose for which a valuation may be undertaken. Reproduction or replacement cost, cost of reproduction less depreciation, etc., all have their proper place, and should be used under the proper conditions.

In order to show this, it has been necessary to go back to the fundamental beginnings of business, as exemplified in the private business, and trace the development of the private corporation and the public utility corporation from this origin, at the same time showing the similarity and dissimilarity between these three classes of organizations. With this as a basis, it is shown how the same principles can and should be applied in the valuation of public utility property as would be used in that of a private individual.

* Presented at the meeting of November 18th, 1914.

This has been done under the three headings: I.—The Private Business; II.—The Private Corporation, and III.—The Public Utility Corporation. The last is again subdivided, and different phases of the subject are more or less fully treated, applying the principles as deduced from the other two divisions.

In investigating the public utility, it is also shown briefly why this class of corporation should be subjected to regulation by the State, this regulation being a necessary result of the circumstances attendant on the development of these corporations from the private business and private corporation.

Inasmuch as the final test of a physical valuation is its ability to stand a review by the Courts of last resort, the decisions of such Courts are referred to in the following pages wherever it has been thought necessary to show the trend of judicial opinion; also, it has been found advisable to quote at times from some decisions, not only for purposes of clearness, but for purposes of criticism.

In conclusion, the writer states that the actual cost to date is a figure equable to the investor, the promoter, the manager, and the public at large. The real object of a valuation is to obtain a figure equable to all parties interested in the particular case under consideration, and every case must be decided on its own merits, and with due regard to the conditions and circumstances immediately surrounding it.

Introduction.

In any work of valuation or appraisal, no matter for what purpose, there are of necessity many perplexing questions that are bound to arise, and their solution will tax to the utmost the experience and resourcefulness of the investigators. On the other hand, there are also fundamental principles governing such work, which, if studied and analyzed, will serve to elucidate many matters which are subjects of controversy at the present time.

In order to arrive at such fundamental principles and obtain a firm basis on which to found axioms for the valuation of the property of public utility corporations, it is necessary to investigate the development of this class of business from its beginnings, and to study it from economic as well as from financial standpoints. To attempt to unravel successfully the so-called mysteries of valuation and ap-

praisal. without such an investigation and without comprehending such first principles, is an absolute impossibility.

One of the principal reasons for so much confusion in regard to certain phases of the valuation of public utility property is that all investigators or appraisers fail to understand clearly wherein a public utility corporation possesses the same attributes as a private business, and, on the other hand, wherein it differs entirely. Therefore, it is necessary to show where the three classes of concerns, (1) the Private Business; (2) the Private Corporation; and (3) the Public Utility Corporation, are similar, and wherein they are radically and entirely dissimilar. In order to do this, and to make matters perfectly clear, it will be necessary to set forth some elementary facts relating to business in general, which perhaps are well understood when attention is called to them, but are forgotten or ignored by discoursers on these subjects.

In the following pages, it must not be supposed that the writer attempts to follow out in full detail all the intricacies of promotion, preliminary investigation, organization, underwriting, financing, or the dozen and one other steps that go to make up the history of any large business enterprise carried to a successful completion. On the contrary, the attempt has been made to describe briefly and in a clear, concise manner the principles that underlie such an undertaking, in so far as they are applicable to the valuation of public utilities. No reference will be made to any company organized simply for the purpose of placing stocks, without the intention of carrying the enterprise to a successful conclusion. It is assumed for the purpose of this discussion that the company is properly organized, is well managed, and that the intention is to return dividends to the investors. It is further assumed that, as far as ordinary business is concerned, there is free and unrestricted competition, and that there is no artificial control by combinations or trusts. Monopoly control is not considered, except in connection with the public service corporation. Such combinations in private business introduce elements into the investigation that are not germane to the scope of this paper.

I.—The Private Business.

Let it be assumed that an individual wishes to engage in a private manufacturing business of some kind, or that he is already engaged

in such. Aside from the transactions incident to trade requirements, the only direct financial dealing that such a concern would have with even a portion of the general public, from which any favors are asked, is the possible necessity of borrowing money from banks. If banks loan money on the concern's notes, such loans are secured by the value of the plant, by the value of the raw or finished material on hand or in progress of manufacture, or by bills receivable, all of which are tangible assets. Other risk of direct loss is taken by those parties selling to the concern raw or finished material on time payments. The notes are soon paid back, the same process being continued over and over again as the necessity arises. Therefore, the risk taken by the general public is practically nil, as it is taken only by those directly interested in the banks from which funds are borrowed, and who, acting through their boards of directors, have taken the chance of the concern not being able to meet its maturing obligations through some unforeseen accident, such as a falling market, financial panic, or similar causes.

In addition, in any dealings with such a concern, a creditor is further secured by the fact that the owner is personally responsible and that all his personal assets are liable as security in case other means fail; in strong contrast to a stock company, in which each stockholder is only liable for the amount of his stock, no matter what mismanagement may have developed or what debts may have been contracted.

If the concern is a co-partnership, the same conditions as those described are applicable, only in this case the risk is apt to be more extended, as there are two or more partners, and, in addition to banks, personal friends of the various partners are more likely to become involved in a failure, thus indirectly leading to a broader dissemination of the results.

If such a concern wished to extend its plant, this would be accomplished in one of two ways: (*a*) the owner would do with less personal returns and thus accumulate a surplus to be used for additions and improvements; or (*b*) he would raise the necessary funds from his banks, with property or other tangible assets as security, or from his personal friends, these loans to be paid back from the profits of the additional business before any benefit would be expected by the owner.

As to the returns to be anticipated from the business, this is simply a question of the law of supply and demand, modified by the ignorance, apathy, patience, desire, or necessity of the community. If the service is not satisfactory, or the grade of goods is not up to such a standard as the purchaser thinks is essential, as there is free competition, he can go elsewhere and obtain what he thinks is satisfactory. If too high a return is demanded, then, on account of the prospects of large profits, still other manufacturers will be induced to enter the field, engender more competition, and so reduce prices. One exception to this would be where some patented or secret process was in use, in which case the only check on the returns to the owner would be the desire or necessity of the public for the commodity, and its ability to pay for it.

As to the physical value of such a business, the owner would naturally carry the actual cost of the plant to date on the credit side of his balance sheet, minus such percentages as he had charged off for depreciation and obsolescence. If a fund for depreciation and obsolescence had been actually placed in trust for the renewal of the plant, it would be a tangible asset of the concern. If not, the owner would have to expect to renew the plant out of his own capital when the time came, just as he had originally built it out of his own funds; or else go out of business at the end of some period with his capital theoretically all returned, plus the amounts that the plant had earned, over and above the depreciation charges, during the years of its operation. It is not to be expected that an owner of such a plant would, under any circumstances, carry its present-day or replacement cost on his books, as such a quantity would have no place on his balance sheet, and, furthermore, would be a constantly varying figure. Neither would such a figure enter into the calculation of profits. Such profits would be based on the actual cost to date, not on any replacement value or present-day value.

If the owner of this business wished to sell it, or if some other party wished to purchase such a plant, the owner would have a perfect right to ask a price such as it would now take to replace his plant, less depreciation, or to erect a new one to produce the same results, which price might be greater or less than the original cost of the original plant, with all its additions and improvements to date. In addition to this purely physical valuation founded on present-day

values, the owner would also ask an additional sum to cover the fact that the business is a "going concern", that is to say, that it is organized, that it has customers, that it has an outlet for its products, and that it possesses the good will of its patrons. What such a figure would be, it is impossible even to estimate. It would depend on such circumstances as (1) the owner's desire to sell; (2) the buyer's desire to own such a plant, modified to a large degree by his ability to build a competing plant if not able to purchase; (3) by the state of the market for the commodity produced, whether rising or falling; (4) by general financial conditions. It is impossible to mention all the many causes, personal, economic, and financial, that might influence such a sale value one way or another.

If, instead of trying to buy the original plant, another party should build a new one and start operations, this new manufacturer would have had to consider (1) the possibly higher cost of the new plant, due to higher material and labor costs; (2) the necessity of building up a new business; (3) possible lessened income due to more competition; (4) lessened income in the early period of development due to faulty organization; (5) necessity of marketing the product; (6) failure due to over-production, etc. All these and many other conditions and circumstances would have to be carefully weighed and investigated before the enterprise was started; but, would the owner of the original plant make any changes in his investment costs? Would he increase the book cost of his plant, because some one else had built a more expensive plant to accomplish the same result? He would simply rest secure in the knowledge that he possessed this advantage over his rival. The new plant, perhaps costing more, must compete with the handicap of higher investment costs, modified perhaps to a greater or lesser degree by more efficient machinery and arrangements than those possessed by the old plant.

If the owner of a private business wishes to retire, there is no restraint to prevent him from so doing. At any time and at a moment's notice, provided he is solvent and has no outstanding contracts that must be fulfilled, he can close his doors, say "I am done and finished", and no one can compel him to continue in business.

The private concern never asks any favors of the municipality or of the State, such as the right to use the streets for car tracks, string-

ing wires, laying pipes, or other uses, or for the building of a railroad or the development of a water power. Nor can it invoke the right of the law of eminent domain to further its own affairs. It is subject to the same laws, rules, and conditions as the entire body politic, and thus, having accepted no special favors from the State, the private concern is not, and cannot be, required to return any special favors to the State.

Such, in brief, is an outline of a simple private business. It must not be supposed, however, that all details are covered or that all incidents which might arise have been mentioned. Any business is susceptible of endless modifications and extensions, and is highly sensitive to all those changes and developments which are constantly taking place in the business world. The outline is given as an exposé of the essential principles of such an organization. Its very fundamental simplicity is to be kept in mind, in connection with the investigation of both the private corporation and the public service corporation.

II.—The Private Corporation.

With the advent of the great stock company, with its large capitalization, its stocks and bonds, and its hundreds and even thousands of stockholders, we come to a new era in the business world, and here begins that intimate relation between the promoters, owners, managers (or whatever other name may be applied to the controlling interests of such a concern), and the general public at large. This relationship passes through all stages of development until the condition is reached where the concern becomes a public utility corporation, managed for and in the best interests of the people at large, who may now have no direct interest in the securities of the company, and yet, at the same time, may control its destinies through commissions and State legislatures.

All private corporations may be grouped in two classes, as follows:

(*A*). The corporations generally known as "close corporations," in which all outstanding securities are held by a few individuals, generally close business associates; and

(*B*). The corporations which, on their organization, offer to the general public stock or bonds, or both, and ask the public to purchase them and become part owners in the enterprise.

In regard to those corporations coming under (*A*), there are three reasons for organizing such a company:

(1) To provide against the disruption of the organization on the death of any prominent member. When a death occurs among the owners of a private business, it generally means the closing and winding up of the firm's affairs. On the death of any one holding stock in a corporation, however, the business is continued as before, under the direction of the board of directors. The securities held by the deceased simply pass into other hands.

(2) To limit the liabilities and responsibilities of those who hold stock in the corporation. As previously stated, the owner of a private business is liable to the full extent of his property, but the holder of stock is liable only to the extent of the amount of stock held by him.

(3) To enable an individual who is a bankrupt, to continue or engage in business, under the name of "manager" or "proprietor," the stock of such a company being held by a few of the "manager's" friends.

The corporations that come under (*A*), as long as they remain in this class, are practically private concerns, and no special attention need be given to them. On the other hand, however, the line between (*A*) and (*B*) is not at all clearly defined, and a company that is classed under (*A*) may at any time come into the other class by offering its securities to the public.

The primary object of organizing a large private company, such as classed under (*B*), is for the purpose of obtaining funds with which to carry out an enterprise of such magnitude that the money cannot as a rule be raised among a few individuals. Such an organization also enables an individual to take an interest in the company, by the purchase of a small amount of securities, he knowing that the amount actually invested by him is the only sum he risks, and that all his assets are not liable. The small investor can thus enter into and obtain the benefits of a large business, whereas if limited to the co-partnership arrangement, he would be unable to do so. In addition, such a company makes it possible for a few small investors, by thus pooling their capital, to engage in a business which otherwise would be controlled only by men of great wealth and financial standing in the community.

After the work of preliminary investigation and organization has been completed, such a company begins its public career by applying to the State for a charter of incorporation, and sets forth in such charter in what ways it proposes to exercise its activities. The promoters, then, on its incorporation, ask the general public to become owners or partners in the enterprise by the purchase of its securities. As it is impossible for all such stockholders to take an every-day active management in the company, such management is vested in a board of direction, the members of which are elected by the stockholders and are answerable to both the stock and bondholders.

The actual cash outlay for the enterprise, including all preliminary work, construction costs, and working capital must be raised by the sale of securities. In order to obtain this capital it is at present the general custom to offer the bonds of the company at a discount, not so much for the purpose of increasing the rate of interest on the investment, which is what such discount really amounts to, but simply to play on the psychological element of the human mind, which wishes to believe that it is getting something for nothing. Where an entire issue of bonds is underwritten by a banking house, or a syndicate composed of several such houses, part of such discount may cover the brokerage fee for handling and placing the securities and guaranteeing the funds when actually needed by the corporation. For example, if an issue of bonds is taken by a syndicate at 90, the syndicate takes the responsibility of placing them among the general public at some figure in advance of this and making the difference as its fee for the work. Such underwriting is more apt to be done with the securities of a public utility than with those of a private company. In addition to this discount on the bonds, a percentage of such purchase price, in stock of the company, is often given to each buyer of the bonds as a bonus, a part or all of which the syndicate may retain as part of its fee.

The promoters and organizers of the enterprise, in order to be remunerated for their work, also expect to be paid with a large block of stock, or with part cash and part stock of the company. In addition, as these organizers as a rule also wish to keep control of the company, enough stock must be issued and held by them so that a majority vote of the said stock will remain in their hands. Frequently, it happens that if a few individuals subscribe to a large percentage

of the issue of bonds, these men will insist on keeping control of the company, and so such stock is issued to them to insure such control. It is thus seen that the average corporation necessarily starts out with a certain amount of over-capitalization, or "water" in its securities, which it is impossible to prevent under present-day laws and conditions. This necessary over-capitalization, therefore, consists of the following:

(1) Promoters' remuneration in stock and cash;
(2) Discount on bonds;
(3) Stock distributed with the bonds as a bonus;
(4) Stock issued to keep control of the company.

On account of the way in which the business of the company is managed (by the board of direction), and because the company is controlled as described by stock issued for the purpose, the small investor, or "minority stockholder" as he is generally called, has very little voice in the real affairs of the concern. The Courts have ruled time and time again, when matters have been taken before them by minority stockholders, that, as a general proposition, in the absence of fraud or special circumstances, it is a question of "the greatest good for the greatest number."

More important, however—and of most consequence to the community at large—is the fact that the security holders of a large corporation are able to go to the banks and borrow cash, giving as collateral these stocks and bonds. Even the concern itself is able to borrow money from the same source, on the stock and bonds that it may have in its treasury. Thus stockholders of the banks and depositors who really may wish to have nothing whatever to do with some particular corporation, are brought into the matter entirely without their knowledge or consent. The financial world is filled with cases wherein banks have loaned to their full capacity on securities on which, in times of financial stress, they have been unable to realize, and have thus been unable to meet the demands of their depositors, although the securities in themselves were good and the concerns behind them perfectly solvent.

The stock company is thus seen to vary greatly from the private business, particularly in the following essential principles:

(1) The real "owners," or those who have actually supplied the capital, may have practically nothing to say about the management of the company, and have no control over their own investment.

(2) The company may have outstanding more or less securities which, having been issued to cover what may be termed "intangible elements," have nothing behind them in the way of tangible assets. When such securities are sold by the original holders, and pass into the hands of the public, there is nothing to distinguish them from those that are supported by tangible assets.

(3) These intangible securities, as well as those supported by tangible assets, may be used, through the medium of the banks, to borrow money from the general public.

(4) Returns in the form of dividends must be secured on a larger amount than the actual capital invested in the actual tangible business, as all securities issued are expected to bring in returns to their owners.

Aside from these matters, and as far as the actual conduct of the business itself goes, the private corporation does not differ materially from the private business, except in details which are not material for this discussion.

It is for reasons such as those briefly outlined that the State exercises control over all private corporations, and justly subjects them to rules and regulations that will protect, not only the public at large, but also those directly interested in the company. Some States exercise such control with a firm, and at the same time just, regard for the rights of all concerned, and others do so in a careless and haphazard manner that leads to fraud and corruption of every description. However administered, such control by the State is an absolute necessity. In other words, the State is beginning that unfortunately necessary paternal supervision over private business, which, later, finds its greatest expression in the direct control of the public utility corporation by the State.

III.—The Public Utility Corporation.

1.—General.—To-day we use the terms, public utility corporation, public service corporation, or quasi-public corporation, without a very distinct idea as to just what is included or should be included by these terms in the way of corporations. With our rapid development, the purely private business of to-day may become the public

utility of to-morrow. Almost everything now classed as a public utility was unknown a short hundred years ago. In ancient and medieval times, great cities existed and flourished, there were no railroads, and steam was unknown. To-day, however, the railroads stand at the head of our public utilities, and how could a modern city exist without them? How long will it be, therefore, before other lines of business endeavor now considered purely private matters will be classed as "public utilities?" As time goes on, more and more concerns, with more and more diversity of products or lines of endeavor, are being brought into this category. Therefore, what is a public utility, and how can it be defined and distinguished?

A public utility corporation, or a public service corporation, may be defined as a private company which exercises its functions in a direction that is of vital importance to the members of the community reached by its activities. Webster says:

"Private corporations, including all others of a civil nature, as ordinary business corporations and those corporations such as railroad companies, lighting companies, water companies, etc., organized or chartered to follow a public calling or to render service more or less essential to the general public convenience or safety, and now often called public service corporations or sometimes quasi-public corporations."

The great distinguishing characteristic of the public utility corporation, which separates it from the private company or private business, is, that it is organized with the idea of working to a certain extent in direct co-operation with the community—and by community the writer means either a village, a city, a State, or a Nation. Such a corporation asks certain definite concessions and grants from the State, as a preliminary to starting in business at all. For example, the railroad asks the right to cross or close highways; to take property from citizens by right of eminent domain (and, of course, for a just compensation); to build bridges across navigable rivers; to narrow the waterway of smaller streams, or even to change their course and direction by means of embankments and fills; the street railway, the water company, the gas company, the electric light and power company, all ask the right to use the public streets to conduct their business; the power development company asks the right to dam rivers and use the water for private gain. Of transcendental im-

portance, however, both to the corporation itself and to the community at large, is that of necessity this class of corporation is a monopoly; if not *de jure,* at least *de facto.*

In return for all these special privileges granted to it by the State, the company promises necessary or useful service of some kind to the citizen. It is the abuse of such privileges on the part of the corporation and, on the other hand, onerous conditions imposed by States on such companies, that have brought about the necessity of just regulation and control of them by the State. That such regulation is equable, few will question at the present time; but how it is to be accomplished, and on what basis, is another matter.

As a basis for the control of any public utility, it is necessary to know the cost, present value, or fair value, of all the property, both tangible and intangible, that goes to make up the utility. Without such knowledge it is impossible to form any definite plans to regulate these companies.

It is in regard to such value that the first serious divergence of opinion occurs among appraisers, and this lack of unanimity exists, not only among engineers, but also among financiers and the judiciary. For example, in making a valuation, should actual cost to date, cost of reproduction new, or fair or market value, be used? Should such matters as depreciation, obsolescence, and inadequacy, be taken into consideration? Inasmuch as the rights and privileges of the company have been conferred directly by the State, should such intangible elements as going concern values or franchise values be considered?

In any valuation work that is to be subjected to judicial review (and practically every appraisal must sooner or later stand this test), it is necessary to give careful study and consideration to Court rulings on the subject, in order that the work may hold through such an ordeal. It must not be forgotten, however, that a Court is compelled to render a decision on the evidence as presented before it. If a case is presented in a different manner, supported by the proper evidence, the ruling of the Court is apt to be very different. The writer's experience in court has shown conclusively that an attorney, although he may have a good case, through lack of proper study and a thorough understanding of its principles, may present it in such shape as to leave the Court no alternative but to give an adverse decision in the matter. The entire subject of valuation and appraisal is still in its

infancy, and the records of the judiciary show that higher Courts have repeatedly reversed the decisions and rulings of lower ones. Therefore, too much dependence cannot be given to judicial precedent in matters relating to public utility valuation and control.

After having given careful study and thought to the matter, coupled with practical experience in this field and an investigation of commercial relations in general, it is the writer's opinion that actual cost to date is the logical and just figure to be used as a fundamental basis for public utility control. Although such actual cost should be used as a basis, it does not follow that this is the final figure that should be used for any and all purposes for which a valuation may be made. The writer does not agree with those who hold that the same ultimate figure should be used, no matter what the purpose of the valuation. The object of the valuation very properly should affect, not only the final figure, but possibly the method of conducting the examination as well.

2.—Valuation for Rates and Security Issues.—In any valuation for the purpose of rate-making, or for regulating the issuance of securities, actual cost to date is the final figure that should be used, provided there has not been fraud, incompetency, extravagance, or anything of a kindred nature in the development of the utility. Such cost should include:

(1) All preliminary work of developing and organizing the enterprise, including promoters' fees.

(2) Preliminary investigations and preliminary engineering work of all kinds.

(3) All final engineering work, including the preparation of final plans, supervision of construction work, etc.

(4) All construction work of whatever nature, up to the starting of the enterprise.

(5) All other overhead charges during construction, which include administration, organization, legal expenses, taxes, insurance, interest, etc.

(6) All additional work of new construction built since operation started.

(7) Where new work has replaced old, and the cost of this new work is greater than that of the old, the difference in cost between the old and the new must be included; but care must

be taken that maintenance is not by any chance classed as new work.

(8) All working capital.

The actual cost should not include:

(9) Anything in the way of excessive promoters' fees or profits, or excessive salaries during construction.

(10) Anything in the way of engineering or construction work that partakes of the nature of fraud, incompetency, or extravagance.

(11) Discount on bonds.

(12) So-called development expenses during the early stages of the operation of the enterprise.

(13) Any part of the development that is not actually used or useful in the operation of the utility for the public benefit.

(14) If new work has replaced old, such as a heavier bridge, or a new cut-off, and the cost of this new work is included in the valuation, under no circumstances must the cost of the now abandoned and useless work be included also.

Many of the most important legal decisions relating to valuation uphold the present or fair value as the equable one to use for either of the above-mentioned purposes. In view of the general increase in construction costs, this practically amounts to cost of reproduction less depreciation. (See Ames *v.* U. P. Ry. Co., 64 Fed., 165; Smyth *v.* Ames, 169 U. S., 466; Willcox *et al.* *v.* Consolidated Gas Company, 212 U. S., 19; Cedar Rapids Gas Light Company *v.* Cedar Rapids, 223 U. S., 670.) The principal grounds on which these decisions are based, and on which the advocates of this system of physical valuation found their arguments, are as follows:

(1) Many of the older corporations, such as the railroads, were organized, and the utility built and operated, before State regulation in detail and public service commission control were thought of, and that, therefore, it is not now equable to deprive these companies of the increased value of the property.

(2) Also, in the case of these older utilities, it is almost impossible to get any idea of the actual cost of the property to date.

(3) The company is entitled to the benefit of any increase in the value of its property since construction, and if such increase is not

allowed, it amounts to confiscation of property within the meaning of the Fourteenth Amendment of the Constitution. Conversely, if the property has decreased in value, the company must stand this loss.

In the case of some of the older public utility companies, there are undoubtedly grounds for the contention that not to allow the increase in value which has occurred would perhaps amount to confiscation. On the other hand, such companies have had a long period of unregulated existence in which returns may have been received sufficiently in excess of what are now considered equable to the community to cover this matter. In order to reduce physical valuation to some standard basis, it may be necessary to ignore some such claims in the future. No change, however slight, can be made in our commercial and financial systems without working some injustice. In a change so potent as that of the regulation of these companies, it would be strange indeed if some unavoidable injustice was not done.

It is also to be remembered that all these old corporations received their rights, immunities, and special privileges directly from the State. It is a fundamental principle of the old English law, that no legislature can limit or restrict the actions of any succeeding legislature. Therefore, any such body can alter or modify the acts of any previous similar body, and can even revoke franchises or other special privileges. Such acts, however, on the part of a legislative body, are in their turn modified and controlled by decisions of the Supreme Court (which have always upheld the Fourteenth Amendment) that property cannot be confiscated, directly or indirectly, but can only be taken by due process of law. (See Stone and others *v.* Farmers' Loan and Trust Company, 116 U. S., 307.)

It must likewise be kept constantly in mind that the price of all material and of all labor entering into the construction of public utilities, has risen in value, with very few exceptions, since the older utilities were built, and, therefore, in spite of improved methods of construction, the present-day value is higher than the original cost. In the case of land, this has often increased in value three or fourfold, and sometimes, especially in the case of city property held by railroads, eight or tenfold. It is for this reason that all public utility companies wish the present-day value to be used as a basis for rate regulation or for the issuance of securities.

Although many decisions uphold present or "fair value" as the proper one to use for rate purposes, there are others that uphold actual cost to date as the just figure to be used. (See San Diego Water Company *v.* City of San Diego, 118 Cal., 556; 50 Pac., 633; Brymer *v.* Butler Water Company, 179 Pa., 231; 36 Atl., 249; Interstate Commerce Commission, in Advances in Rates, Western Case, 20 I. C. C. R., 307.)

In addition to these decisions, if some of those which uphold present and fair value are examined, places will be found which indicate that the Court is not at all sure of its stand. For example, in Smyth *v.* Ames, Justice Harlan said:

"And in order to ascertain that value, the original cost of construction, the amount expended in permanent improvements, the amount and market value of its bonds and stock, the present as compared with the original cost of construction * * * are all matters for consideration * * *."

In this decision the fact of original cost being an important element in an appraisal is admitted.

In Ames *v.* Union Pacific Railway Company, Justice Brewer said:

"Nevertheless, the amount of money that has gone into the railroad property—the actual investment, as expressed, theoretically, at least, by the amount of stocks and bonds—is not to be ignored, even though such sum is far in excess of the present value."

This opinion needs no comment.

In Willcox *v.* Consolidated Gas Company, Justice Peckham said:

"If the property, which legally enters into the consideration of the question of rates, has increased in value since it was acquired, the company is entitled to the benefit of such increase. This is, at any rate, the general rule. We do not say there may not possibly be an exception to it, where the property may have increased so enormously in value as to render a rate permitting a reasonable return upon such increased value unjust to the public."

Justice Peckham here lays stress on what he terms "the general rule", but the question naturally arises: What is this general rule, who made it, and why should it be used as a basis of a decision by the Supreme Court of the United States? The Justice further says, that if the value has greatly increased, it might be unjust to the public to base rates on such greatly increased value; but, if this

would be an injustice to the public, why would not a rate based on any increase in value be unjust? Who is to decide in such a case where to draw the line between such increase on which rates would be just and where they would be unjust?

Another argument against the use of this increase in value is that a public utility corporation does not go into business with any idea of speculating in its investment, any more than a private manufacturing concern does. The increase in value has not been caused by any direct efforts on the part of the company, and amounts to an "unearned increment" on which the public should not be required to pay rates.

As to the impossibility of obtaining the actual cost of a utility, this contention is also doubtless true in the case of some of the older concerns, but the writer thinks that the difficulties are greatly over-estimated in many cases. In any event, this is no excuse for putting all other utilities in the same category. If the actual cost to date is available, or if it can be obtained with reasonable efforts, to ignore this value and appraise the property on any other basis, such as reproduction cost, is to enter unnecessarily into the realms of speculation and surmise, which is so severely condemned by Mr. Justice Hughes in the Minnesota Rate Cases, in referring to land values (Simpson *et al.* *v.* Shepard; Same *v.* Kennedy; Same *v.* Shillaber, 230 U. S., 352).

Some few years ago the writer was chief assistant in an important investigation and appraisal of a large private industrial plant, which work was used in a suit in the United States Circuit Court involving a claim of $30 000 000. In this case the original cost of the plant was available, but it was necessary to examine carefully the actual property, and check conditions as they existed with the actual cost of the work. In addition, it was also necessary to follow back commercial conditions for the nine years previous to the appraisal, and discover what results would have been accomplished under such conditions. Of course, this was a long and tedious process, and absolute accuracy was essential on account of the legal aspects of the case. There was no reason, however, why these conditions could not have been traced back twenty or thirty years as well as nine; and, if the actual cost of the plant had not been available, it would have been perfectly feasible to appraise it at any period in the past (within a

reasonable time), using such prices and values as applied to that period. In this case, the present or fair value of the plant would have had no possible significance.

The actual cost to date represents the actual amount of capital invested in the enterprise; it is the amount on which the investors expect returns, and, if the enterprise is a success, on which they should receive dividends. This figure is also a constant quantity, it only being necessary to correct it for additions made from time to time; whereas, the present value (or value obtained by any other method) is a constantly changing quantity, it being necessary to alter it as conditions and prices fluctuate.

In order to demonstrate one phase of this problem, let it be assumed that the cost of reproducing a utility would now be less than when the plant was placed in operation. If the State or the Court should say to the corporation: "You must reduce your capital under the conditions as they now exist", the writer fails to see why this is not confiscation of property within the meaning of the Fourteenth Amendment. It is not just thus to wipe out invested capital, as Justice Brewer pointed out in Ames *v.* U. P. Ry. Co., 64 Fed., 165.

One of the arguments against using the actual cost under such circumstances is the claim that there are many utilities, especially railroads, which could not charge a high enough rate to pay a return on the investment without working a palpable injustice to the public; and the familiar illustration given is that of a branch railroad built to tap a logging or mining district, where, after the natural resources are exhausted, there is not enough traffic to pay operating expenses, aside from interest on the investment. This is all true, but a problem of this kind should be treated in an entirely different manner. Such a road never is, and never would be, constructed in the same substantial manner as a permanent line built for all time. The entire line and equipment would be of a temporary nature. When such a road is contemplated, it is the promoter's business to take such facts as its temporary character into consideration, and a return must be obtained during the years of successful operation, such as will, at the end of its period of usefulness, have returned the capital to the owners, plus all expenses and a fair rate of interest on the invested capital; and under these circumstances such rate of return should be allowed by public service commissions. In other words,

the entire capital of the railroad must be amortized by the time its period of usefulness is over. If, however, a road has been built, and through unforeseen conditions has ceased to pay a return on the investment, then it becomes a simple question of an unsuccessful business enterprise, with bankruptcy, a receiver, and a winding up of the company's affairs. This is the way a private enterprise would be treated, and there is no reason why a public utility should not be handled in the same manner.

A further objection to this method of treatment may be raised, on the ground that the utility is of absolute necessity to the community, as, for example, a water company supplying water for household use and fire protection. Such an objection is well founded. However, a community cannot compel a company to operate indefinitely at a loss, so, under such circumstances, the community should operate the utility at its own expense until such time as a proper sale value can be placed on the property and the utility is acquired by the community, such value to be founded on "market value", or, in other words, the "bankrupt sale value", and not on either actual cost to date, replacement value new, or any other value.

In many appraisals it will be found that there are often items which, if a fictitious value such as present-day cost is given to them, will lead to embarrassing and often ridiculous results, whereas, if their actual cost is used, there is no chance for an injustice, either to the company, to the rate-payer, or to the investor. The following are given as some typical examples:

According to an old print, the great Tring cutting, made on the line of the London and North Western Railway in 1837, was excavated by hand and the spoil was removed in hand-barrows guided by men and pulled up inclines on the sides of the cutting by ropes passing over pulleys and worked by horse-power. If such a piece of work was to be valued by cost of reproduction, should the same conditions be duplicated, only with present-day prices for labor and material, or should it be assumed that the most modern methods would be used, with steam shovels and construction trains? In either event, it would mean entering into the realm of speculation and surmise, when the facts are there before the investigator.

On many of the early railroads built in the United States, temporary trestlework was constructed of timber obtained along the line.

It surely would not be just to estimate a value for such works based on present-day figures for labor and material entering into them.

In developing a water power, it is common practice to obtain the timber for the coffer-dams, and also the sawed and dressed lumber for form work or other uses, from the forest immediately contiguous to the site of the development. The writer knows of cases where such sawed lumber has been supplied at from \$10.00 to \$12.50 per m. b. m. Later, when all this timber has been cleared off, it would not be equable to estimate such lumber at a price of from \$25 to \$30 per m. b. m. just because it would now cost the latter figures to obtain it at the site. On the other hand, the cement used in many of the masonry dams in the Western States was obtained at an exorbitant figure. As lines of transportation develop, and markets come nearer these sites of power development, there is no reason why the value of the dam should be placed at a lower figure, because cement can now be obtained cheaply.

Several years ago the cost of a certain class of steelwork, erected in place, was 4.07 cents per lb. Since that time, this price has not been reached, but the same class of work has been done as low as 2.87 cents per lb. (an exceptionally low figure). This is a difference of 29½ per cent. It would not be just to compel the railroad which had paid the higher price to reduce its capital to suit some arbitrary figure which would be a mean between these two extremes. The capital for this work was honestly invested, and no part of it should be wiped out.

In regard to those items, mentioned in the first paragraph of Section 2, which should and should not be included as capital in a physical valuation (numbered from 1 to 14, inclusive), it is necessary to discuss some of these more in detail.

Referring to (1), "Promoters' fees", it can readily be explained why this item should be taken at actual cost. This fee depends on the circumstances immediately surrounding the specific case under consideration. What would be excessive under some circumstances, would not be commensurate in others. One of the largest hydro-electric developments in the world was advocated for more than 10 years by its promoters before final completion. Such devotion to an ideal is worthy of more substantial reward than if 6 months or 1 year only was spent in such work.

In some cases the promoter's fee may also include and cover all preliminary work of developing and organizing, in which case the one item only must be included in the valuation. For these reasons, if this item is estimated on a cost-of-reproduction basis, it amounts simply to a guess as to what is equable.

As to those items included in (5), "All overhead charges during construction", there is no way by which they can be estimated on a present-value method with any degree of accuracy. The system usually adopted is to calculate such items as a percentage of the inventory reproduction cost of the utility. Such a figure might be very much less or very much greater than the actual cost. The amount of each of these items depends on the particular enterprise, its period of construction, and the conditions immediately surrounding it. Therefore, they should be included at actual cost only.

In regard to interest on capital during construction, there is one element of this matter which the writer does not recall having heard mentioned by any author on this subject, and which tends to reduce the total interest payable before the completion of the enterprise. This is that final payments on contract work are sometimes not made until the utility is in operation. The writer has drawn up many contracts for construction work on public utilities in which the following clauses were inserted:

"The company shall, however, deduct and reserve ten per cent. from any periodical payments falling due for work done and materials furnished hereunder, to be paid only at the expiration of 30 days after the works shall have been completed, approved, and accepted by the engineer * * *."

"The works shall not be finally approved * * * until all machinery and apparatus furnished hereunder shall have been regularly operated in the manner for which the same are intended to be used for a period of 30 days from the date of starting the same in regular operation * * *."

Referring to Item (7), cases may occur where the new improvements possibly cost less than the original investment. Any cases such as these must be cared for by a fund to cover depreciation, obsolescence, and inadequacy. Referring to the private business, it was seen that such a concern usually charged off a certain percentage of the cost of the plant from year to year, until the capital invested was theoretically all returned. As a public utility corporation cannot do this,

and go out of business at any time at its own volition, the securities covering any such abandoned or superseded property must be retired by withdrawals from the fund set aside for that purpose, unless there are other tangible assets not covered by securities to take the place of the useless property.

Taking up those items which should not be included as capital, and referring to Item (9), it has often been the case in the past that the promoters of the enterprise received such a large remuneration that the company was strangled with overhead charges before it had a chance to start. Such excessive fees must not be included in any physical valuation, and only a fair amount must be allowed under such circumstances.

Thus any excessive salaries of officers or of those in charge of the construction of the utility must not be included. If there has been anything of this kind in the development of the enterprise, only such an amount must be included for this item as such first-class direction and supervision could have been obtained for in the open market.

"Discount on Bonds", Item (11), must not be charged to capital. This item, no matter in what light it is considered, amounts to an increased rate of interest on the capital actually invested. As interest is considered an operating expense, and a sum is set aside from gross receipts to take care of it, so discount (which is only deferred interest) should be taken care of by equal increments set aside from gross receipts from year to year. That is, discount must have been amortized by the time the bonds mature.

Even if the discount includes the brokerage charge, it is still interest that must be met, and so should be amortized as explained previously. The brokerage charge is payment for the use of money; therefore, it is a true interest charge and should be treated as such. All commission rulings in regard to this item follow this method in dealing with the question.

Referring to Item (12), it will be noted that early development expenses are excluded. The reason for this is that, according to the writer's view, these are strictly operating expenses. If a utility is properly managed, the development stage never ceases. The training and education of employees for their various duties never end. The advertising department is always attempting to obtain more and

more patronage for the utility. As to errors of design and construction which may develop during the early days of the operation of a utility, and the correction of which so many appraisers claim should be paid for out of capital, these also should be paid for out of profits, if there is money to pay for the work, because there is practically no limit to which such changes can be carried. For example, on railroads expensive tunnels, long cut-offs, great trestles, deep fills, and heavy bridges, are to-day being constructed simply to correct such errors of design and construction; and if such corrections and betterments are to be capitalized as new development, what security is left for the obligations already issued against the original and now abandoned or superseded work? In other words, "development" should be the watchword of every utility, whether in economy of operation, increase in business, or increase in satisfaction of service. The appraiser who attempted to include any such items as early development expenses in a physical valuation, either on the basis of actual cost to date, or present value, would be confronted by a peculiar problem. He would not know where to begin or where to leave off. Even if detailed accounts of the company were open to him, he would be no better off. It would simply be a matter of his personal opinion as to what items of this class of operating expenses he called development expenses, and what he called operating expenses, and his personal opinion, also, would alone be his guide as to where in the corporation's history he would stop charging development expenses to capital and charge them all to operation.

Furthermore, relative to this matter, it is to be remembered that most utilities have been slow in growth, and that it has not been necessary to train forces or build up an organization from absolutely ignorant material to handle the complicated conditions of to-day. For example, in the railroad world, the transition from the small systems beginning in England in 1825 with a few miles of track, tiny cars, and 8-ton locomotives, to the great transcontinental lines of to-day, with thousands of miles of track, cars of 110 000-lb. capacity, and great Mallet compound locomotives, was not made in a single stride. It was a slow development, and the corporations gradually expanded and improved to meet the conditions as they arose.

In regard to Item (13), if a utility has been constructed on a larger scale than is necessary to supply the service properly, then the cost

of such excess size must not be included in the valuation. There is an exception to this, where a utility has made provision for future extension and development to a limited degree; but it must be only to a limited degree. If the utility has made such provision out of all proportion to the needs of the present community, then it cannot expect to obtain rates on the cost of the same. The capital must have been wisely and judiciously invested. (See Smyth *v.* Ames, 169 U. S., 466; Pioneer Telephone and Tel. Co. *v.* Westenhaver, 29 Okl.; 118 Pac., 354; San Diego Land and Town Co. *v.* National City, 174 U. S., 739.)

3.—Real Estate.—In regard to the valuation of real estate, a number of elements enter into this problem, which differ from those affecting the valuation of other tangible property, and, for this reason, this item is generally considered separately.

All public utilities have the privilege of resorting to the rights of the law of eminent domain to acquire property, if such property can be shown to be necessary for the proper development of the enterprise. Thus, an individual, although he may foresee that his property is going to enhance greatly in value in the future, can be forced to part with it on the basis of its value at the time condemnation proceedings are instituted.

Those who uphold the present-value theory of valuation maintain that land should be included at its present value, as determined by the price of similar property in the immediate neighborhood, plus a percentage to cover the expense of what it would now cost the utility to acquire such land. That it costs a utility much more to acquire real property than its actual market value, is now such a well-known fact as to need no argument here.

Such extra cost in the case of urban property consists of legal expenses incident to the transaction, plus the costs of condemnation proceedings, if final resort is made to the law of eminent domain. Also, more particularly in the case of a new utility, there may be the cost of damages to adjoining property, caused by the very fact of the utility being in operation at all. In the case of country property, there is also the cost and value of such elements as temporary and permanent severance damages, plottage, and continuity (the three latter items applying more particularly to railroads). As all such extra values and costs are known to the average seller, he asks such

a price, over and above the fair market value of the land, as he thinks the corporation will pay, without resorting to condemnation. As time is of very great importance, and condemnation is apt to be a slow process, the corporation is generally willing to pay even more than this would cost.

For city property, the percentage of increased cost to the utility runs from 120 to 300%, and, in the case of country property, from 200 to 800% of the fair market value of such property without improvements.

In discussing this question, the following facts are to be kept in mind: In the case of many utilities, especially the railroads (our leading public utilities), land was purchased, even in the cities, at a very low figure. In many cases, large grants of land were made to the railroads, sometimes by individuals and communities for right of way and station sites, and by the United States Government for similar purposes. In addition, in some cases, large tracts were also given by the Government simply as a bonus for building the railroad at all. All such grants were made on the understanding, if not specifically stated, at least implied, that the corporation was to return useful and satisfactory service in some form to the community.

Almost without exception, the value of such land, whether obtained by purchase or gift, has increased enormously, such increase having been caused in large measure by the very fact of the utility being there at all. All utilities now claim that they should be allowed to include in a physical valuation all such land, not only at its increased value, but also at a value obtained by increasing such present market value by the same proportion as the original value of the land was increased in cost to the utility. In other words, the utility corporation wishes to put all conditions back as they would be without the utility, except the greatly increased value of the land, then assume that the utility wishes to acquire such land, and must go through all the processes to do so, increasing the value of the land by the percentage as already determined, and this to apply whether the land in question was obtained by donation or purchase.

To make this perfectly clear, assume the following concrete case. Land at a value of $100 per acre was acquired by a utility corporation at a total cost of $250 per acre, or 2½ times its market value, and adjoining land of the same value was also donated to the company.

All such land is now worth $1 000 per acre. The contention of the corporations is that all such land, including that donated, should be included in a valuation at a price of $2 500 per acre.

Even if an appraisal is being made on a present-value basis, the writer fails to see any logic or justice in such a contention. If the land is to be included at its present value and such value is high enough to cover all the original cost of acquiring it by the corporation, then no multiplier whatever should be used. In other words, the utility corporation, in the case of land, wishes to assume a set of theoretical conditions that could not by any possibility exist, and then found value on such conditions. It is these very elements that are so severely criticized and condemned in the Minnesota Rate Cases (230 U. S., 352) and that were decided against the utility corporations.

As a further example, suppose a community has presented a station site to a railroad. The corporation claims the right to ask rates on the value of such property and to issue securities against it; but, as rates are supposed to be a return on capital invested, and as securities are supposed to be issued to obtain capital to build the road, how, by any possible process of logical reasoning, can donated property be used for such purposes?

How untenable this present-value theory of land is can be demonstrated by illustrating an extreme case. Assume that a railroad had acquired terminal facilities in a city years ago, when land in the immediate vicinity of such property was cheap and unimproved. Now, suppose that the neighborhood improves, that the city grows up to and surrounds the terminal, and that the adjoining property becomes improved with expensive hotels and office buildings (this assumption is a fact in many cities). It would surely not be equable to value such terminal land for rate or security issue purposes on the theoretical supposition that it was all, or any part of it, occupied with such buildings, and that the public utility corporation (the railroad) must now pay the full value of such land, with all the improvements thereon to date, in order to acquire it. On the other hand, if, in order to provide for necessary extensions, the corporation was compelled to purchase any such land with such improvements thereon, then, even though the improvements were destroyed, the land should be valued at the total cost to the company.

Therefore, land or real estate should be included in any physical valuation on the same basis as any other property.

4.—Depreciation, Obsolescence, and Inadequacy.—In discussing the matters of depreciation, obsolescence, and inadequacy, most appraisers seem to take the stand that some definite scheme must be decided on at the commencement of the operation of the utility, and adhered to without any change, by which its total value will be returned to the owners at the end of some fixed period, which is as near as may be the end of the useful life of the plant. The following theories have been developed to produce this result:

(1) Replacement method for depreciation;
(2) Sinking-fund method for depreciation;
(3) Straight-line method for depreciation;
(4) Equal-annual-payment method for depreciation.

It is not necessary to discuss these methods separately. The trouble with all of them is, that, in order to work them out and discuss them at all, it is necessary to assume conditions in regard to the life of the plant and its operation, which, in practice, from the very nature of a public utility property, cannot be realized. These assumptions are as follows:

(1) That the physical plant of the utility, or at least its component parts, have a definite life;
(2) That the value of the plant must have been returned to the owners at the end of this period;
(3) That the owners must reinvest this returned capital at the end of the period in a new plant.

These assumptions are fundamentally wrong. It is impossible to give exactly the life of any parts of a utility; therefore, it is impossible to set aside or accumulate exact amounts to replace them. The best that can be done is to assume some term of years, based on former experience with the physical element under consideration. In addition, new developments, discoveries, and inventions, may, almost over night, cause valuable machinery and equipment to be consigned to the scrap heap, and thus the element of obsolescence comes into being. Conditions may change slowly or suddenly, so that the equipment of a utility becomes inadequate to handle the conditions as they

now exist. Thus, inadequacy is "developed". As an example of the former, the machinery of the cable roads in New York City had to give way to electric traction long before it was worn out. To illustrate the latter, there is the necessity for larger cars to handle the increasing traffic in large cities, or the necessity of railroads replacing wooden cars with steel, before the former have served their physical period of usefulness.

If the value of the plant was to be returned to the owners at the end of its period of usefulness, there would be a time, between the end of the old plant and the beginning of operation of the new, when there would be no utility. A utility, however, cannot go out of business of its own volition. It must continue to give good and useful service to the community (see Weatherly *v.* Capital City Water Co., Ala. 22 So., 140; Judge Haight in People *ex rel.* Manhattan Ry. Co. *v.* Woodbury, 203 N. Y., 231; 96 N. E., 420).

Therefore, in regard to depreciation, obsolescence, and inadequacy, such elements in the maintenance of any public utility should be taken care of by a fund maintained for this express purpose. This fund should be accumulated from a yearly increment taken from gross receipts and charged to maintenance costs. With a very few exceptions, every element in a utility begins to depreciate from the moment the construction forces place it in position. In a general sense, obsolescence and inadequacy are also depreciation. The utility should be kept up to date and up to 100% efficiency and value by constant withdrawals from this fund, to make good the three classes of deterioration previously mentioned. Therefore, at no time should such a fund amount to a very large percentage of the cost of the plant.

In order to maintain the fund in an equable manner, the amounts placed to its credit must, as far as possible, be evenly distributed over the years of the operation of the utility. There is no reason, however, why such amounts may not vary from year to year within reasonable limits, or why some definite plan should be fixed on and adhered to for a term of years, in order to accumulate such a fund, without any regard to conditions and circumstances which may develop or change in regard to the actual physical property.

It is impossible always to have an exact balance between the actual depreciation on the one hand, and the amount of the fund on the other. The fund, however, should be kept as near as may be to such

an amount. If such a fund is actually kept in trust for this purpose, there is no question but that it should be treated as an item of the plant, and rates should be allowed on it. However, if no such fund exists, or if it exists only in theory on the books of the corporation, then, in a valuation of the utility, depreciation may become a very important item, and must be given careful consideration. This may be treated in the valuation in one of two ways, which lead to the same result: The property may be appraised at its full cost, without depreciation, and returns may be allowed on this figure, no special mention being made of the depreciation fund; or the property may be appraised at its cost, less depreciation, and returns may be allowed on this and also on the depreciation fund.

That utility corporations are expected to keep the plant in first-class condition by constant withdrawals from some such sinking fund or its equivalent, and without allowing any excessive accumulation of such a fund, is set forth in Knoxville *v.* Knoxville Water Co., 212 U. S., 1; Cumberland Telephone and Telegraph Co. *v.* City of Louisville, 187 Fed., 637; and People *ex rel.* Manhattan Railway Company *v.* Woodbury, 203 N. Y., 231.

5.—Going Concerns and Franchise Values.—There are no other matters in the appraisal of a public utility that are so difficult of a fair and equable solution as the values to be given to the intangible elements, going concern, and franchises. More difference of opinion exists among appraisers in relation to the method of treatment of these items than in regard to any others of the entire subject, some maintaining that no value whatever should be placed on them, others saying that as well not value the utility at all, as to omit them.

As to going concern value, there is nothing tangible that has ever entered into it, except the general management of the company. On the other hand, however, it is an admitted fact that the mere cost of the bare physical property of a utility does not represent the full value of a concern that is organized, managed, and operated in an efficient and satisfactory manner.

Various attempts have been made to arrive at some equable method of obtaining an equivalent tangible value for this intangible element, the best known of which is the capitalization of earnings over and above a fair return on the actual invested capital. In a physical valuation for the determination of rates, as the going concern value

deduced by this method depends on net income, and net income depends on rates, which is the very thing that is to be determined, it is impossible to obtain any such value in this way. Any attempt to deduce going concern value from rates, or to found rates on going concern value, is simply reasoning in a circle.

If this question is referred to the private business for study and comparison, the writer thinks that an equable solution of it may be deduced; and, as far as he knows, such solution has never been propounded before.

The private individual never charges up any value for going concern on the credit side of his balance sheet. What he does is this: In order to determine a price at which he can dispose of his commodity profitably, he first figures the total actual cost to produce the article (including amounts to cover depreciation, etc.), or, in other words, the "operating cost". The next item he adds to this cost is a fair rate of interest on the actual cost investment in the plant, and then follows an item to cover the general risk of the business. Lastly, he would add a figure to cover the fact that he is in business at all, or, in other words, that his concern is a "going" one, with all the advantages that this term implies. This, coupled with the risk item, is his profit in the venture.

There is no valid reason why the returns to the owners of a public utility corporation should not be determined in the same way. The operating expenses, including an allowance for depreciation of all kinds, are known or estimated, the physical cost has been determined, and a rate of interest must be allowed on this tangible value, such as the same amount of cash would return if placed at ordinary rates of interest. In addition, a further rate must be allowed to cover the risk of the enterprise.

Now, coming to the fact that the utility corporation is a going concern, in exactly the same way as the private enterprise is, there is no reason why the return for this intangible element cannot be equably obtained by adding some percentage to the figure already determined to cover the items before mentioned. In other words, these two items—the risk and the going concern—are the "profits" of the enterprise, and are so intimately related in practice as to be almost incapable of separation. In one sense, ordinary interest cannot be considered a profit, as such return could be obtained without any

investment in the utility. If there is no going concern, there is no profit, and, therefore, the return for going concern is covered when a fair rate (profit), over and above enough to cover expenses and ordinary interest, has been allowed.

Objections may be made to this method on the ground that it is impossible to determine any standard figure by which to gauge all cases in which going concern value must be determined, and that it really amounts simply to the opinion of the appraiser as to what to use. Such an objection, however, applies with equal force to any system whatever for estimating the value of going concern. In any event, the determination of a figure is simply a matter of the personal opinion of the appraisers as to what is an equable value for this element, and it is a much simpler matter to decide on a percentage of the actual physical investment in the plant, to cover it, than to go through an intricate set of calculations which are purely arbitrary and only tend to befog the real issue in the case.

If a valuation is made for the purpose of issuing securities, inasmuch as securities are issued, at least theoretically, to obtain capital to build and operate the utility, and going concern value cannot enter the problem until after the utility is in operation, therefore, no consideration should be given to this item for such purposes.

It is often assumed that franchise value and going concern value are so closely related as to make it impossible to separate one from the other for purposes of valuation. There is no doubt that these two elements are very intimately connected, but, at the same time, the writer thinks that there is enough material difference to make it very apparent why the two should be separated for purposes of discussion. A franchise is simply a permit, or the grant of the right, to conduct the business at all. On the other hand, going concern value is a value resulting from the use and application of the rights and privileges granted by the franchise. A concern may possess a franchise, which in itself is valuable, but on account of mismanagement and incompetency, the business may have no going value.

If a franchise only runs for a limited number of years, it has less and less value the nearer its time of expiration approaches. When it has expired, of course it has no value, although, under these conditions, the Courts have held that there is still a going concern value

in the property. (National Water-Works Co. *v.* Kansas City, 62 Fed., 853; Omaha *v.* Omaha Water Co., 218 U. S., 180.)

Furthermore, there may be a tangible element of value that attaches to a franchise from the fact that money may have been expended in a perfectly legitimate way to take the necessary steps to obtain it; or a franchise may have been purchased from a municipality, or by one company from another company. Thus, there may be a tangible element to take into consideration in the valuation of a franchise that does not occur in going concern value.

Although all franchises were granted by the State, the consensus of all Court rulings is that, once the grant having been made, and the corporation having expended capital under the implied protection of such grant, the State cannot revoke the franchise without giving to the corporation full and just compensation for its value. Just what value this is, no one has yet been able to determine. Every case has simply been decided by recourse to some arbitrary figure.

As far as rates are concerned, as the franchise is only the permit to conduct the business, if a fair return has been allowed for risk and going concern, that is all that is just and equable, and no special provision should be made for the franchise value, except in so far as actual expenses have been incurred in obtaining the same.

6.—Valuation for Purchase or Sale.—If a utility is to be valued for the purpose of purchase or sale, either between companies or between a company and a community, in the absence of a special agreement to the contrary, the present or fair value (in the majority of cases, cost of reproduction less depreciation) is the just and equable figure to be used for this purpose, except as hereinafter noted. Actual cost should be used simply as a guide in arriving at this figure.

By referring back to the paragraphs dealing with the sale of a private business, and applying the same reasoning to the utility corporation, the writer thinks that the justice of this method under these conditions will be perfectly apparent. The security holders have a perfect right to have the full value of the plant, as it stands at the time of sale, returned to them, as this is the figure it would cost the purchaser to reproduce it. Conversely, if the plant is now worth less than when it was built, the purchaser is entitled to the reduced price, as he could now build a similar plant for this figure. No purchaser has any grounds to say to a seller that he must part with his

goods at a less price than they would bring in the open market; nor has the seller a right to expect a higher price under similar conditions.

There are some exceptions to this use of the present value for some items, in cases where a community is the purchaser and the seller is a corporation that has exercised its functions under a charter granted by said community. These items are those of going concern and land values.

In regard to going concern value, the doubt as to the justice of allowing an amount to cover this item under the foregoing conditions arises from the fact that the community generally claims that, the franchise having expired, or being about to expire, and the corporation being about to go out of business, there is no value left for going concern. On the other hand, the corporation claims that, as the community is going to obtain a plant fully developed and in full operation, it is very different from starting up a new enterprise from the beginning, and that there is, therefore, a perfectly legitimate value to going concern.

The truth of the matter is that both parties to the controversy are partly right and partly wrong, and the only equable solution is to follow a middle course and allow some value for the going concern, but not such a large figure as the corporation would undoubtedly ask. Following such a course is the way the Courts have attempted to decide this difficult question (National Water Works Co. *v.* Kansas City, 62 Fed., 853).

As to land values, here also, under similar conditions of sale, it may not be equable to charge the full reproduction cost. The reasons for this in the case of land are perhaps not as apparent as in that of going concern, but the following are set forth as some limiting considerations:

If a utility is being appraised for purposes of sale, it may be that although the land now has a high value, other land could be obtained for the same purpose at a greatly reduced price, that would be just as satisfactory; but, as the utility cannot be moved without the destruction of the plant, it is impossible to separate the land from the utility. Furthermore, the land may have been obtained by recourse to the law of the right of eminent domain, and owners thus forced to part with their property against their will, when they may have realized that they were losing a good investment. It does not seem

equable that they or their immediate descendants should be compelled to pay a greatly increased price under such conditions.

In a case such as the foregoing, if the sale was one between private interests, it would simply mean that the buyer would go where cheaper land could be procured, if an exorbitant price were demanded by the seller. Both the buyer and seller are free agents, and are not compelled to enter the transaction against their will. In the case between the community and the public utility company, however, as the latter is and has been a monopoly, and for good and valid reasons either one or both of the parties may have been compelled to enter the transaction against their will, the solution of this part of the problem must be treated along somewhat different lines from that of other tangible property. There is only one seller and only one buyer, and no open market.

Under such circumstances, each case must be solved according to its own merits. It would seem here, as in the case of going concern values, that a middle course must be followed which will give as equable a solution as possible to both parties.

7.—*Valuation for Taxation.*—For the purposes of taxation, it does not make any difference what value is used, if the utility is under full State control, except that it might favor one section of the country as against another—this applying more to the railroads than to any other utility.

This can be explained as follows: The utility being under State regulation, all operating expenses are allowed over and above any return on the investment, and taxes are classed as an operating expense. If a local section of a road is assessed at a high valuation for purposes of taxation, and most of the revenue is obtained from through traffic, this traffic would then have to pay these taxes, to the benefit of the locality; but if the utility is a local one, there is no difference one way or another. The taxes come from the people in the form of operating expenses, and are returned to them as taxes, be the assessed value high or low, or the taxes on the assessed value high or low.

IV.—Conclusion.

There are some phases of the public utility corporation that the writer has not considered it necessary to discuss, inasmuch as they have no direct bearing on the valuation of the property of these com-

panies. Such, for example, is the company organized simply with the idea of acquiring and holding the securities of public utility corporations, now known as a "holding company", which class of concern is at present being severely criticized by the public and the States. A number of questions, also, that are being debated at the present time, such as the agency theory as applied to corporations in their relations to the State, and whether a public utility has the right to invest surplus profits, or a depreciation or amortization fund, in new construction and additions to the plant, and then charge rates on such investment, have not been touched upon. In addition, some physical elements, such as piecemeal construction, solidification and adaption of roadbed, pavement over mains, working capital, etc., have not been discussed in detail. The reasons for these omissions are that it is impossible to discuss every question that may have a bearing on the physical valuation of the property of public utility corporations, within the limits of this paper.

Furthermore, no attempt has been made to quote all legal decisions that may apply to the subjects discussed, nor has any attempt been made to give decisions in support of all contentions and arguments. Only enough have been given to show the general trend in the more important cases.

As was stated in regard to the private business, so also every public utility is capable of endless modifications and variations in its details of organization, the conduct of its affairs, and its relations to and with the State. When all is said and done in regard to the control and valuation of these corporations, every case must finally be decided on its own merits, and with due regard to the conditions and circumstances immediately surrounding it.

The real object of a valuation of any public utility is to obtain a figure such that any calculations or regulations founded on it as a basis will be just to all parties that have any interest whatsoever in the matter.

In the foregoing pages, the private business, the private corporation, and the public utility corporation have been outlined and compared; and, based on the similarity and the dissimilarity of such corporations with the private business, it has been shown how the property of these companies may be valued for different purposes. Broadly speaking, as far as the actual physical business is concerned, there is no difference

between the public utility corporation and the private business, and it is for this reason that a physical valuation of the property of a company of this class should follow the general lines that a private business would use in its accounting systems. There is no reason why there should be any difference in this respect between the two, except where elements are introduced into the problem that arise from the monopolistic character of the public utility.

On the other hand, however, it has been shown how the public utility corporation as an organization is, in its very inception, entirely different from the private business and the private company, and, on account of these methods of organization and development, how its relations with the State are of necessity on an entirely different status from those of the latter concerns. These main points of difference are here summarized as follows:

1.—The public utility corporation, before engaging in business at all, must obtain special favors from the State.

2.—It enters into direct business relations with the State.

3.—It therefore becomes the agent of the State.

4.—After having once entered into such an agreement with the State and accepted the trust, although it now has invested its own capital (property) in the enterprise, it cannot withdraw from the agreement on its own volition.

5.—Such invested property is held under different conditions and circumstances than that invested in private enterprises.

6.—It is a monopoly granted and, with certain limitations, upheld by the State.

7.—It is for these reasons that the State has the right to exercise detailed control over the public utility.

Therefore, in making an appraisal and valuation of the property of public utilities, if the points of similarity between this business and the private business and private corporation are kept constantly in mind, it will be seen why actual cost to date is the basic figure to be used in all cases, as has been set forth in the foregoing paragraphs; and, if this figure is used, it eliminates, as has been shown, many of the elements in such work that seem to be so difficult of solution. Actual cost is a figure equable to the investor (the real owner), to the promoters, to the managers; and last, but by no means least, such figure is absolutely just and equable to the public at large.

DISCUSSION

ALEX DOW,* M. AM. SOC. C. E. (by letter).—Mr. Gandolfo, after noting the existence of a lack of unanimity among engineers, financiers, and the judiciary, asks: Mr. Dow.

"For example, in making a valuation, should actual cost to date, cost of reproduction new, or fair or market value, be used?"

Why the question? A valuation is a valuation, and a statement of cost is a statement of cost. The two are not the same, and there should be no confusion between them in the mind of an engineer. A statement of what a property has cost can be prepared by a competent accountant from the books of the company, unless the books have been wilfully falsified. A valuation is an engineer's work, and the manner of making it is properly a subject of discussion by this Society. The problem is: Given a certain property, required a valuation. The answer is: An itemized list of the property on an assigned date, and the value thereof, as of the same date, as found by the competent engineer.

It may be that the engineer's instructions are that he shall determine the cost of a certain property to its present or past owners. In that case, no engineering knowledge of values is required, or of use, except as a check on the accuracy of the cost figures shown by the record; and an engineer accepting such instructions presumably either has himself the necessary knowledge of accounts, or will secure competent assistance for the fulfillment of the instructions accepted. Instructions to make a valuation or appraisal, however, do not require, except collaterally, the ascertainment of cost, and instructions to ascertain cost do not call for valuation or knowledge of values. Why the confusion between the two which seems to exist in the author's mind and in those of many other engineers?

On page 855, Mr. Gandolfo says:

"In any valuation for the purpose of rate-making, or for regulating the issuance of securities, actual cost to date is the final figure that should be used."

How is the purpose of a valuation related to the ascertainment of value? Or is it related at all? In such talk, as to the purpose of valuation, are we not departing from the engineering standard which requires an engineer to tell the truth as he finds it, without fear or favor? Are we not setting up a false standard permitting that an engineer's opinion of value shall be different when his client proposes to buy, from what it is when he proposes to sell? Is it not the duty of an engineer called on to make a valuation to say that he finds certain property the present value of which is thus and so? If the en-

* Detroit, Mich.

Mr. Dow. gineer's opinion is asked as to what would be a wise price to be accepted or offered for that property; or if he is appointed arbitrator to fix a price; or if he is asked to express an opinion as to a fair rate of return to be allowed on the property in a rate-making case—under any of these conditions, an opinion as to price or as to rate of return is proper; but, to inject his opinion as to price or as to rate of return into his findings of value is to depart from his direct function as an engineer telling what he has found and at what he appraises its present value for its use or purpose; and, to confuse opinion as to price, or as to rate of return, with value of property as it exists, is darkening of counsel.

In the writer's opinion three-fourths of the existing confusion tending to discredit engineering reports of public utility values arises from the failure (or the refusal) to recognize existing value and recorded cost as two distinct things; the other fourth arises from permitting the interest or intention of the client to influence the report of the engineer as to findings of fact. The way out of the confusion is: First, to recognize the fact that value is value and cost is cost, and that the setting of a rate of return or a permissible price is a function outside of, and beyond the making of, a valuation, or the ascertainment of cost; and, second, to recognize and observe fully the distinction between an engineer's appraisal of value as found, and the opinion or argument of a retained expert.

Mr. Lavis. F. Lavis,* M. Am. Soc. C. E.—The avowed purpose of this paper is to demonstrate that "the proper figure to be used [for the valuation of public utility property] is the actual cost to date", questions of fraud, extravagance, mismanagement, etc., being supposed to be eliminated.

The speaker does not agree with this, and, considering the very able paper by Mr. Alvord,† it hardly seems necessary to go into any very extended argument to attempt to show that the "actual cost to date" method is impractical, and that the "cost of reproduction" method appears to be the only one which most nearly complies with all the necessities of the situation.

Mr. Alvord points out the necessity of keeping clearly in mind the distinction between cost and value, and it seems to the speaker that as far as any part of the discussion may be applied to the valuation of steam-operated railways, and particularly to those engaged in interstate commerce, it would appear at once that a valuation based on the actual cost to date method, even though it were possible to apply it, would by no means necessarily give a true estimate of the "present-day value."

One must also not lose sight of the fact that, according to most of the Court decisions, and for the purposes of most valuations, the

* New York City.

† *Transactions*, Am. Soc. C. E., Vol. LXXIX, p. 117.

present-day value is further limited and defined by the requirement that this shall be "the value of the property for the purpose for which it is being used", and it is keeping this in mind which makes it sometimes difficult to see why depreciation should be deducted from the reproduction cost of a machine which is working at 100% efficiency. Mr. Lavis.

Of course, we need not lose sight of the fact that much assistance can be derived from knowledge of actual costs in determining present-day values, or the fact that it is the duty of those charged with the determination of present-day values to use such information for all it is worth. Every railroad engineer of experience, however, knows the difficulty, not to say in many cases the absolute impossibility, owing to the lack of records, of determining for many—even for most—of the important railway systems what the actual cost to date is or what the actual cost of even a reasonable percentage of its component parts has been.

Take, for instance, the very simple and common case of a road which had originally been built as a single-track line and afterward widened, without introducing any questions of changes of gradient or location, though these have almost always been present to add an additional complication. Widening a cut of a line already in operation may cost more for the excavation than it would to have taken it out altogether in the first place, by reason of interruptions of traffic and other difficulties, or it might cost less, by reason of the fact that the presence of the existing line afforded better and easier facilities for doing the work. The value of such a cutting to-day, however, is not changed in any way by the fact that it was all taken out 20 years ago, or part of it 20 years ago and part of it 10 years ago, though these facts most probably did affect its cost.

It is quite true that individual cases might be cited to show that in instances similar to that just referred to the actual cost to date method might be a true measure of the present-day cost, but that hardly alters the fact that, in general, it probably is not, or at least not necessarily so. Arguments can be advanced and concrete instances quoted to show the injustice, under certain conditions, of any method of valuation as applied to individual cases, but, admitting that valuations of railways have to be made, and especially looking at the matter from the standpoint of the Interstate Commerce Commission, some general method must be adopted which will be equally applicable and fair to all the railways of the country, to determine with a reasonable degree of accuracy and fairness their present-day value, which may or may not be the actual cost to date, and most likely would not be.

So many arguments and concrete cases bearing on this phase of the subject have already been published in the *Transactions* of the Society,

Mr. Lavis. that it seems only necessary to refer briefly to the matter in this discussion in order that Mr. Gandolfo's statements may not become part of its official records without some protest.

Mr. Harte. CHARLES RUFUS HARTE,* M. AM. SOC. C. E. (by letter).—In endeavoring to establish a foundation for his proposition that "in any valuation or appraisal that is to be made for purposes of rate-making or security issues, the proper figure to be used is the actual cost to date of the work in question", Mr. Gandolfo points out "differences" between the private business, the private corporation, and the public utility corporation, which, at least to the writer, seem of degree rather than of kind.

Proportionately, the private business deals as extensively with that portion of the public in its field as either of the other groups; its commercial paper stands on the value of the business, including the exceedingly important intangibles of good will and business reputation, precisely as the securities of the others stand or fall—a glance at the proceedings of any Court of Bankruptcy will show how often misjudgment of these factors has caused failure, and the "compositions" effected testify to the intangibility of much, if not all, of the assets; the liability is proportional to the holding in any case—the sole owner, whether he represents a private business or 100% of corporation security, is liable in full; and the frequency with which private business expansions are made on personal or firm notes which lack support and are not "paid back from the profits of the additional business before any benefit" is "expected by the owner" keeps many a lawyer busy.

No small part of the failures which not infrequently occur when old and apparently prosperous concerns attempt to expand, is due directly to bookkeeping which has figured profit on an original cost instead of the greatly increased value of the plant, and predicated the expected return from the expansion on that basis.

That "the private concern never asks any favors of the municipality or of the State", is a statement hardly borne out by the facts. Much the larger portion of the occupancy of the highway is by the teams of individuals, no less by favor of the municipality than the use for tracks, wires, pipes, or the like, of the public utility corporation; and no small part of the time of the common council or its equivalent is taken up with the consideration of petitions for marquee, area-way, connecting bridge, or wire and pipe location rights by individuals.

It is true that the private concern cannot "invoke the right of the law of eminent domain", but, for that matter, neither can a large proportion of the public utilities. That right is not, as the author seems to intimate, a special privilege to a public utility "to further its

* New Haven, Conn.

own affairs", but, on the contrary, is a State prerogative to protect the public against the individual who, otherwise, might block needed improvement; it is delegated only to further that end, and, in its exercise, the public necessity must always be specifically shown. Mr. Harte.

The reasons given for organizing a private corporation are equally cogent in the foundation of a partnership, which is nothing more than a corporation of special form; indeed, the individual owner of a business is the limiting case of the corporation.

The most important reason of all for incorporating is only half touched on; modern business demands such large capital that few care to put so large a part of their money in one investment, but would rather divide it among many projects, in order to reduce the extent of the loss in case of failure of one or more. Such scattering of investments, however, does not relieve the investor of one whit of his responsibility, which is exactly in proportion to his holding.

The "essential differences", as summarized by the author, hardly seem to be such. All the real owners of the utility may not control, but the majority do; the minority are in the same position as the minority in any republican form of government, and the aggrieved stockholder has the same right to invoke the protection of the Courts—and uses it, too—as has the taxpayer who is dissatisfied with the action of his governmental body.

The other three "principles" are as characteristic of private business as of corporations, whether private or public.

With the author's general idea of a public utility, that it does a business essential to the public welfare and from its nature—the great plant required, the physical exclusion of a competitor, the necessity of centralized control, etc.—is essentially a monopoly, there can be little dissent, but its difference from the other two groups is only in the fact that, monopolizing a public necessity, it is held subject to public regulation purely as a matter of protection, and not because of special privileges enjoyed; by far the greater part of the "Sherman Act" cases decided or under way are against businesses which by their expansion are alleged to have monopolized the market and thus to have become public utilities, although enjoying no rights other than those of a private business.

In his argument to the main question, Mr. Gandolfo states that the decisions in favor of "reproduction less depreciation" are based on three arguments:

1.—Inequity of depriving old companies of increased value in property;

2.—Impossibility, in the case of older companies, of determining cost to date; and

3.—The protection afforded by the Fourteenth Amendment.

Mr. Harte. A more correct statement would have been restricted to the third reason, together with the interpretation by the Courts of this Amendment. Until the Constitution is further amended, "property" is protected against confiscation without compensation; until another amendment, or the reversal of many and consistent decisions of the Supreme Court and the establishment of a new ruling, the value of property is its worth at the time in question. So firmly is this established as the "general rule" (and for the benefit of the author, who is apparently unfamiliar with this expression, it might be said that it is a common law term meaning the established judicial attitude toward the question at issue), that we find Justice Holmes saying:

"It is no longer open to dispute that under the Constitution what the Company is entitled to demand in order that it may have just compensation is a fair return upon the reasonable value of the property at the time it is being used by the public." (San Diego Land and Town Co. *vs.* Jasper, 189 U. S., 439, page 442.)

The "actual cost to date" method fails to give recognition to that which the engineer should be particularly jealous to safeguard, for it is that alone which warrants his professional existence—scientific knowledge.

The intelligence that foresees the future industrial center in the wilderness of the present; the knowledge and courage that in the face of all but insuperable difficulties carry through to successful completion a great development—these which are really the chief elements in creating the real value of the utility, are entirely ignored by the "cost to date" method, which, by accepting the wasteful and unwarranted expenditures of the dreamer and of him whose daring is that of ignorance because the investment was made in misguided honesty, puts a premium on incompetency and extravagance.

In the author's assumption, on page 860, he apparently confuses property and cost. The property is measured by the value, and not by the cost, of what is held, and it is the value, not the cost, to which the protection of the Courts is extended. The position of the Courts is very clearly set out by Justice Hughes:

"It is clear that in ascertaining the present value we are not limited to the consideration of the amount of the actual investment. If that has been reckless or improvident, losses may be sustained which the community does not underwrite. As the company may not be protected in its actual investment, if the value of its property be plainly less, so the making of a just return for the use of the property involves the recognition of its fair value if it be more than its cost." (The Minnesota Rate Cases, page 46, Senate Doc. No. 54, 63d Cong., 230 U. S., 352.)

The reason for this is well founded. The public, in constructing its own and competing plant, would have to pay the prices as of the

Mr. Harte.

date of work; it would enjoy the benefit of advances in the art; it would have to bear the burden of increased costs over those of an earlier time. It is obviously as unjust to compel the public to pay rates on a cost in excess of what it would have to pay to reproduce the facilities as it is to compel the utility to make rates on a cost less than that which would be required if the public either bought the old plant at a fair value, or built a new one.

To use the original cost of the Tring Cutting—aside from the fact that, unless there is much more and better data than that of "an old print," any estimate of the original cost would be far more speculative than one based on modern methods—would deprive the public of the benefit of progress in excavation apparatus and methods; to hold the early-built railroads to the very low cost of trestling built from timber cut along the line would deprive them of an element of value, in many cases actually paid in by the company through renewals of the old structure with new materials at the higher later price, which, affording no betterment, must be charged to operation.

The author refers to the holdback as tending to reduce the interest charge. As a matter of rigid accuracy this should be considered, but the actual amount involved is small, with the entire project covered by contract, and with the construction funds drawing full interest up to the time of their use, there would be a saving, on one-tenth of the cost, of one-sixth of the rate of interest. Usually, however, the company purchases the bulk of the material, while the construction fund, because of its activity, rarely commands one-third of the ruling interest, and often less than this, making the net saving due to the holdback about one-fiftieth of 1 per cent.

The writer is not in accord with many of Mr. Gandolfo's propositions, but his treatment of depreciation and its allies, in the clear recognition of the distinction between accounting depreciation and actual depreciation, seems to be one of the few rays of bright light in the fog in which so many writers have concealed the facts.

It is in the consideration of "Valuation for Purchase or Sale" that, in the writer's judgment, the error of the author's ways becomes obvious.

Let us consider a property the returns of which have been fixed by the cost to date, but having, by virtue of wise handling, a value 50% greater than its cost. B buys the property at its value; can there be any question as to his absolute right to jump the rates 50%, the cost to date having so jumped without one whit of change in service?

Or, if the contrary condition obtained, B buying for 50% less than cost, could he prevent a rate reduction of 50% although he gave exactly the same service as his predecessor?

Mr. Harte. The fact is, that, as the author himself says, "With our rapid development, the purely private business of to-day may become the public utility of to-morrow." There is no hard and fast line of distinction, either between the two classes of business, or between the principles and practices applying to them.

Unless the owner be permitted to earn at least a fair return on the capital which his investment represents—and that is obviously what it would bring in a free sale—there is confiscation; to base rates on cost to date, when the investment in a free sale would bring much less, is unfair to the rate-payer; it would seem that the Supreme Court, in its consistent stand for value to date, had logic with it.

Mr. Green. F. W. GREEN,* ASSOC. M. AM. SOC. C. E. (by letter).—Valuation is a subject which is rather "meat for strong men", than "milk for babes". To be an expert, one must not only have the ability to use the yard-stick of the engineer, but that of the lawyer, the publicist, and the economist, as well. The subject looms larger, perhaps, in its economic aspect than in any other. Any valuation, or system of valuation, based on principles which fail to satisfy the requirements of law, of economics, and of engineering, is, *ipso facto,* to be rejected.

It is but natural that there should be divergent views on a matter so involved. In writing about it, we are all apt to reflect unconsciously our hobbies and our prejudices, and these are often as likely to be wrong as to be right. Although the writer cannot agree with the author in many of his conclusions, yet he believes the paper offers many opportunities for profitable discussion, and elucidates and clarifies the subject in many particulars.

In all the discussion regarding valuation, the writer fails to recall an instance in which attention has been called to the radical difference between electric, gas, and water utilities, on the one hand, and steam railroads on the other. When the entire output of a utility is one thing—water, gas, or electric current—the units of service are homogeneous and commensurable. If we seek fair rates for service to the consumer, we may write the equation:

Gross Earnings minus the sum of Operating Expenses and Taxes, Reserves for Depreciation, and Fixed Charges, equals Fair Rate of Return multiplied by Fair Value.

From this we may write:

Fair Rate for Service equals the product of Fair Rate of Return and Fair Value plus the sum of Operating Expenses and Taxes, Depreciation Reserves, and Fixed Charges, divided by Units of Service.

Per contra, the units of service produced by the steam railroad are neither homogeneous nor commensurable; and unless we are prepared to say that a ton-mile shall be a ton-mile, whether the commodity

* Stamps, Ark.

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be suburban passengers, immigrants, iron ore, straw hats, coal, or silverware, either car-load or less than car-load, and that a uniform rate per unit of service shall apply uniformly and invariably, it would seem to the writer that some of the theories of valuation as applied to common carriers must fail. Valuation is not herein deprecated; it is believed that it will do more to dispel the misapprehension and prejudice, which have been created and fostered by certain persons with political ambitions, than any other single cause; but, at the same time, it would be well to point out this condition now, lest some of us be disappointed hereafter.

On pages 847-848 the author betrays a mistaken attitude as to eminent domain. He states: "Nor can it [the private concern] invoke the right of the law of eminent domain to further its own affairs." History teaches the error of this concept. Eminent domain, the prerogative of Government, exercised only in and for the public interest, was delegated first to certain canal companies, and later to railway corporations. In the delegation of this power, the Government of every State, as far as the writer can learn, specifically restricted its exercise to purposes in which there were real and vital public interests, and these had to be proven before the right could be exercised. A number of cases may be referred to in any good law library, in which it has been held that the right of eminent domain may not be exercised without restriction, or merely to serve a private interest, for instance, for a track to a private industry, as a rock quarry, manufacturing plant, etc. When such right is exercised by a carrier, it is not by virtue of any inherent right vested in the carrier *per se*, but rather because it is more expeditious and convenient for the carrier as the agent *ad hoc* of the Government to exercise the right, always and only, for the benefit of the public and with ample protection of the rights of the private citizen whose land has thus been appropriated to public use. Otherwise, if there were no such delegation of governmental authority to carriers authorizing them to use, within prescribed limits, the governmental right of eminent domain, the Government itself would have to exercise the right, and, later, after having acquired the land expropriated, turn it over to the carrier.

If one will but take the time to examine the numerous citations pertinent, in a judicial frame of mind and without partisan prejudice, this view finds confirmation in many various phases. For instance, in Louisiana, a case is on record in which it was held that one road could condemn a right of way over and along the right of way of another road on the grounds that although the old road owned the land and was using it for right of way, yet it was to the public interest that the new road be not obstructed; and as the old road had been permitted the exercise of the governmental right of eminent domain,

Mr. Green. in the interest of the public, it could not use the benefits acquired thus, to obstruct the new road in the exercise of a similar right; and all because *salus populi suprema lex est.*

As to the author's reference to watered stock (page 851) it seems to the writer that this is really nothing more than faith—"the substance of things hoped for, the essence of things unseen." If a farmer acquires land for $10 000, and refuses to sell it for less than $20 000, has he not watered his value 100 per cent.? But, does he get any more per bushel for the corn raised on this land?

If banks are not well managed, the remedy is to change management when bad loans are made on inadequate security (page 851). And (page 852) the doctrine of *caveat emptor* applies in the purchase of railway securities, as fully as in all other human activities. The Government which, with a noble compassion, and an attitude of exalted altruism, should undertake to protect fools from their folly, would have a bigger job than the Panama Canal.

Our friends who devoutly espouse Socialism, believe in the public control and management of all the utilities of production. To the writer it seems that the author's definition (page 853) of a public utility, would cover a bakery, a haberdashery, or a "movie"; for what are more vital to the community than bread, clothing, and recreation? Although it is necessary, of course, that certain public utilities be regulated in the public interest, it would seem equally necessary that we do not permit ourselves to drift into the conception that we should have one kind of law for certain enterprises, and another kind for others.

The author's questions (page 854) as to what items should be taken into consideration are answered in the first sentence of the succeeding paragraph; but the answer is apparently disregarded in his advocacy of the principle of actual cost to date. The writer cannot agree with the author (page 855) that "The object of the valuation very properly should affect, not only the final figure, but possibly the method of conducting the examination as well." If a valuation is real, it should be the same for rates, taxes, issuances of securities, purchase, or sale. Otherwise we should have the ludicrous condition of a high valuation for taxation, and a low valuation for rates, as was the case, a few years ago, in Texas.

The argument against the allowance of "unearned increment" seems to be inconsistent. In the case of a company deferring the purchase of land, say, 20 years, until needed, the author would allow "actual cost"; but if the land were acquired by the company at the beginning of the 20-year period for, say, 10% of the amount paid at the end of that period, the author would allow only "actual cost". In the first instance, he would pay the "unearned increment" by including it in

his valuation; in the second case, he would not allow it. From the standpoint of the rate-payer, why should he pay more in one case than in the other? If the "unearned increment" is justly due to the virtuous citizen, so also to the vituperated corporation. Mr. Green.

On page 865, the author criticizes roads which now find it necessary to adjust their plant to present traffic, by various corrections in grade and alignment; but, in the second following paragraph, he would not include "excess size" of plant in his valuation. How such treatment of the subject could be called a "valuation" is hard to conceive. A corporation that would continue as such, if thus treated, would certainly be a wonderful thing to contemplate.

JOSEPH MAYER,* M. AM. SOC. C. E. (by letter).—Mr. Gandolfo's paper represents a widespread state of feeling toward public utilities, which influences the action of many voters and public men. The idea that compensation should be in proportion to the effort made to secure it, is the leading motive of his paper, and of the bulk of the socialistic working classes, as well as of many of the most unselfish philanthropists. Mr. Mayer.

In the competitive system of the production of wealth, actual profits of different industrial enterprises are hardly ever average profits, and, instead, vary widely, from losses of part or nearly all of the invested capital to very large annual returns, even exceeding the investment.

These differences sometimes result from unregulated natural monopolies (and have then often little or no connection with superior industrial skill), but more often from differences in the industrial talents of the leaders of the various enterprises. These differences are purely mental qualities, which cannot be measured directly, but can only be judged from the revenue which they produce.

The promoters, managers, and owners of such enterprises consider them as tools for obtaining a revenue, and they endeavor to create these tools at the smallest possible cost. The value of the tools depends on the revenue they secure. Though their cost of production, including average profits, has some influence on their value, the most cursory observation of the actual returns secured on the invested capital of different competitive enterprises shows that there is no definite or close relation between the cost of production of industrial enterprises and their revenue and value. Therefore, those who attempt valuations of complicated enterprises by ascertaining their cost of production meet with various unsolvable problems, especially when they try to ascertain the proper compensation for the mental work of their leaders. They often complain of the excessive compensation of the promoters and financiers who are the middle men between the investing public and those who first suggest or conceive the enterprises.

* Montreal, Que., Canada.

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One of the most difficult and important tasks in the creation of an enterprise is to ascertain whether it should be created, or whether its value when finished will equal or exceed its cost of production. It is evidently desirable to use the available resources for the production of the most useful enterprises. The main function of the middle men between those who first suggest an enterprise and the investing public is to suppress the unprofitable and advance the profitable enterprises. Bankers who offer to the public such securities as prove profitable investments gradually acquire customers who will buy the securities they offer, guided mainly by reliance on the bankers' judgment and honesty. If the bankers possess the ability to judge correctly the value of the securities, or the income which they will bring, and the current relation between revenue and market value, and if they issue them at a profit and below their market value, they perform efficiently the most important and difficult function in the creation of industrial enterprises. These bankers either buy these enterprises from the promoters and then sell them to the public, or they sell their services to the promoters for a commission. These services essentially consist in investigating the enterprises offered, in thereby forming an approximately correct idea of their cost and future net income, in ascertaining the commercial value of the securities issued, and in offering them to the public with their approval. The promoters take up ideas suggested by inventors of such enterprises, assist in the creation of detailed plans and estimates of cost and revenue, and in obtaining the favorable opinion of trusted and well-known men of good judgment on such matters. They then present to the bankers the facts and arguments which assist the latter in forming an approximately correct opinion of the cost and probable revenue.

The attempt to fix a proper compensation for either the promoters or bankers by prescribing a percentage on the cost, or a salary which should not be exceeded, must inevitably fail in properly adjusting it to the value of the services rendered. This value depends mainly on the intelligence, experience, honesty, and reputation of the workers, and only to a slight extent on the work spent on the task in hand. Any meddling with the compensation secured under free competition, beyond insisting on complete publicity and the honesty of all the accounts, must inevitably be rather a hindrance than an aid to the attainment of justice. Practically the same applies to attempts to prescribe the salaries of the leaders of industrial enterprises. The supreme aim governing all the activities of the creation and management of a competitive industrial enterprise is to secure the largest revenue at the least cost; and of thereby making the difference between the value and the cost of an enterprise as large as possible. As long as there is free competition for the purchase of materials and the

services of men, and for the sale of products, the compensation of the owners and the leaders is kept within just limits, and the widely different profits of the enterprises reward those bond and stock buyers who used their resources in a judicious manner for the welfare of the community, either by forming a correct opinion of the reliability of the judgment of others or by following their own good judgment; and punish those who wasted their resources on enterprises which would have been better prevented. If the profits were the same for all investors no adequate effort would be made to choose profitable and avoid unprofitable enterprises. The public would then have to pay average profits on the investments in injudicious enterprises, and would get the excess over average profit of the extremely small number of judicious ones which, under such conditions, would have any chance to succeed. The present liberty of investment would become utterly impracticable, and a public authority would have to be created to select the enterprises to be undertaken. Exactly the same holds true for every important step in the management of an enterprise in operation. Public management would be inevitable. Mr. Mayer.

The competitive system without efficient public control has produced in the course of time values which have, especially for old enterprises, no definite relation whatever to their costs of production to date. To value enterprises which have grown up with little or no public control on the basis of their cost of production to date would cause such a wholesale confiscation of the real value, dependent on revenue, of some, and such an enormous increase in the value of others, that the resulting storm of protest would sweep any commission attempting it into disgraceful oblivion. To imagine that any Court would sustain such a valuation exceeds the capacity of any one with the least knowledge of the nature of competitive business. The author thinks that an arbitrarily chosen or guessed at going value should be added to the physical value in order to obtain the total value of an enterprise. Why not guess the total value without going to the trouble of ascertaining first the physical value? Equally reliable results could be obtained thereby.

It is evident to all competent observers that the necessity of public control of prices arises from the disappearance of competition. Whether the monopolists are private persons, partnerships, or stock companies with few or many stockholders, is entirely immaterial. The author neglected to give any definite advice in regard to the control of issues of securities. He evidently intends to make the securities cover the actual costs incurred, but his description of how to do this is partly indefinite and partly unreasonable. He claims to show that cost to date should govern rates, provided there has been no fraud, gross mismanagement, or extravagance.

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This cost, however, is to be only the basic figure and, as he asserts later, a guessed at going value should be added. The writer is of the opinion that Mr. Gandolfo has not proved his case.

He also claims to show that the same principles should be applied to the valuation of the property of a private individual. What sane buyer or seller of any business will be governed by its ancient history, except in so far as it throws any light on its future earnings? All sane transactions of sale between competent judges of values, able to follow these judgments, which alone throw a light on the real values, are governed by the benefits the owner can derive from the property after buying it. The author, however, is obsessed by the theory that the intelligence used in the creation and management of an enterprise deserves no consideration whatever as a cause of value. He also neglects to describe clearly the method by which cost to date, which is an important fact in ascertaining the rate of profit on the investment obtained by an enterprise, can be ascertained correctly.

Although the writer believes that the paper, as a logical product, is without value, it expresses an influential state of public feeling which demands a reduction of the prices of monopolized products and objects to the persistence of a condition which gives to the investors in monopolies larger profits than are obtainable in competitive enterprises; any argument or utterance which promises some way out of the present condition is welcomed by a large class of readers, because it agrees with their feelings. A way out must be found to obtain justice and thereby to satisfy those who keenly feel that the present condition is unjust. The past franchises are mostly indefinite and often unduly favorable to the owners of monopolies, but they often contain a clause prescribing that the rates must be reasonable. Reasonable rates are evidently such as will give to the owners of monopolies the same profits as are obtained in competitive enterprises, and a regulation of the prices which secures this result is just, where such a clause exists and where it has not become obsolete by neglect to enforce it. The terms of the franchise, therefore, must be consulted, and if they are not obsolete, what has been promised governs the regulation which fixes the rates and the consequent value of the monopoly. The public, however, has the right to expropriate from the owners by paying them the value of the monopoly resulting from this regulation.

If another regulation than that originally intended has been legally introduced and maintained, then the value of the monopoly, with this other regulation legally in force, must be paid to the owners. What kind of regulation is legal at present is a question for judges and legislatures to decide, and is a necessary preliminary to any valuation of natural monopolies. After this legal regulation is known, and has been in force for some time, the probable future net earnings

with it can be judged from the recent ones and the causes of change. Mr. Mayer.

The owner of the monopoly owns the future net earnings, with the present legal regulation. These future net earnings have a definite present value which can be ascertained as soon as they and the proper interest rate are known. The proper interest rate is that rate which makes the present value of the future earnings of similar enterprises equal to their market value. There are some enterprises which have a fair market value and approximately known future earnings; the proper interest rate, therefore, can be calculated. The present fair value of monopolies, therefore, can be closely estimated without any circular argument, and without the use of any arbitrary guessed at going value.

After the present fair value of the monopolized enterprises has been thus ascertained from the estimated future revenue, with the present legal regulation, it is possible to establish such a regulation as will give to the value thus ascertained, and to all future investments, merely competitive profits, by a method previously fully described by the writer.

The author's notion as to how profits and values are ascertained in a competitive business appears preposterous to any one familiar with competitive enterprises. The products of such enterprises are sold at what they will bring in the market. The difference between the price obtained and the cost of production is the net revenue secured. The manufacturer endeavors to obtain with his plant the largest possible net revenue in excess of interest on borrowed money. He will extend or diminish the scale of production of the different articles or services he furnishes so as to secure the largest possible net revenue. He may go out of business when he finds he can obtain better total compensation for his time and property by selling his business and doing something else, but as long as he is in business he has no choice except to take what revenue he can get, which, especially if his business is old, is almost altogether independent of its cost to date. The value of his business is evidently the present value of the future net revenue which can be obtained from it. This will be estimated from the recent net revenue, less a proper salary for a managing owner, and the causes of future changes of net revenue, among which the cost to date, if the business is old, is one of the least important. This competitive method of ascertaining the prices of the products and the value of a business, which any one not quite ignorant of competitive enterprises will recognize as the actual one, is evidently radically different from that advocated by the author for public utility corporations.

When a nation, a state, or a city, engages in business, it is, like any other owner, interested in the profits or losses resulting therefrom. The cost to date must then be ascertained to find the rate of profit

Mr. Mayer. on the investment. This rate of profit secured during a short interval of time, however, cannot be ascertained accurately. The depreciation must be considered as a part of the operating expense which is subtracted from the gross earnings to obtain the net earnings. The actual depreciation of any part of an enterprise which occurs during its life, is its original cost less its salvage or scrap value when discarded. The distribution of this depreciation over its life is at best arbitrary. The depreciation during any one year, therefore, is unknown, even after the article has been discarded.

The depreciation of all the parts still in use is known still less accurately because their life is unknown. The remaining value of an old part without scrap value when discarded, which renders equally good service during its life, is often given as its probable remaining life, divided by the probable life of the part when new and multiplied by the original cost. This is not true, because it neglects interest, which is a very important factor, especially if the life is long. It is also never true that a part renders equally good service during all its life. Inadequacy and obsolescence enter as other unknown factors in depreciation. As the actual depreciation of the parts of a business enterprise during any one year cannot be ascertained with accuracy, the same is true of its net earnings. When, in ascertaining the investment, depreciation is only subtracted when a part is discarded, the investment is over-estimated by the amount of depreciation of all the parts still in use. Although, theoretically, depreciation should be charged when it occurs, practically, whenever there are no expensive parts, it is often best to charge it when the parts are discarded, as then the depreciation of the discarded parts is first accurately known. When comparing different enterprises, it is important to use in all the same method of allowing for depreciation. The author's remarks on how to ascertain the cost to date are rendered quite indefinite, as are all his reasonings, by lack of clear definitions of any of the terms he uses.

That the value of an industrial enterprise is not given by either the cost to date or the cost of reproduction less depreciation, is perceived by all close students of the problem, and becomes apparent whenever the attempt is made to define the terms clearly, and describe accurately either method, and to apply it to concrete cases.

Franchise value and going value are the expedients used to bring the proposed methods into agreement with the facts.

When a franchise gives the right to charge rates which give more than competitive profits, it has a value. This value is the present value of the future excess over competitive profits which the franchise allows. It must be estimated, therefore, by studying the terms of the franchise, ascertaining the consequent excess of future profits

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over competitive ones, and their present value. To make such an estimate possible, the franchise must define the regulation. If it can be shown that a franchise is against the public welfare, it may be invalidated by the Courts. The value of a franchise, therefore, is largely a legal question. It is not equal to the cost of obtaining it, as the author intimates.

Going value has been defined as the difference between the value of a business and its physical value. Some other method of finding the value of a business than the cost to date or the cost of reproduction less depreciation, therefore, must be found. This method, more or less hidden by complicated verbiage, is to capitalize the revenue.

The following equations summarize the method: If T is the total or market value, G the going value, and Ph the physical value, then

$$T = Ph + G \qquad (1)$$

where

$$G = T - Ph \qquad (2)$$

From Equations (1) and (2),

$$T = Ph + T - Ph \qquad (3)$$

or,

$$T = T \qquad (3)$$

The argument is: To find the total or market value, add the going value to the physical value, Equation (1); to find the going value, subtract the physical value from the total value, Equation (2); and to find the total value, add and then subtract the physical value from the total value. This is the method followed in substance, with much intricate phraseology, in a recent paper in a prominent economic journal. Mr. Gandolfo rightly prefers to guess at the going value. By this process, it is always possible to obtain any desired result, and, therefore, it is a very convenient method.

The problem of the valuation of natural monopolies and the regulation of the prices of their products is of the utmost importance at the present time, and the engineering societies can render a great public service and raise themselves in the public estimation by discussing it intelligently and thereby contributing to its solution. Outspoken criticism of popular errors is a necessary part of such discussion, and, the writer believes, should be without reserve when one succeeds in getting a hearing before a scientific society.

The writer is of the opinion that this paper does not show the essential realities of the problem, that its definitions are not clear, and that it does not give an accurate description of any method of valuation.

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J. H. GANDOLFO,* ASSOC. M. AM. SOC. C. E. (by letter).—In replying to the discussion of his paper, it is the writer's intention to treat the subject first in a general way, and then take up each contribution separately, making such comments on the salient points therein as seem necessary in each individual case which has not been covered in the general discussion.

Few people, and much less engineers, particularly those who have discussed this paper, seem to realize the changed conditions surrounding public utilities, and public utility properties, that exist to-day, compared with what they were in the past; and it is not necessary to state that such changing conditions are probably destined to continue to a greater or less degree. Only a few years ago the regulation of a public utility by the State, in the smallest degree, was considered an attack on individual liberty and the right to hold property. As for an appraisal by the State, and the regulation of rates and security issues, such things were looked on as preposterous, and nothing short of revolutionary. Now all is changed, and we find the public utility, as a general rule, accepting all these things with equanimity. The public utility has become almost if not entirely the agent of the State, and as such is answerable in detail to the State.

From the writer's knowledge of the conception and development of private enterprises, and his connection with public utility developments, he fails to see any valid reason for not conducting and developing a corporation, whether a private or public utility, according to the same general principles of honest business endeavor as a conservative and carefully managed private business. There is no reason for having one set of morals or code of ethics for the private business man, and an entirely different set for the corporation and the men who are responsible for its being and management.

The following is an example of what has been considered perfectly legitimate financing in corporate development. An appraisal of an industrial plant with which the writer was connected, and an investigation of the general business and financial conditions surrounding it, disclosed the following condition:

Actual cost of plant, including all buildings and machinery..............	$1 100 000
Appreciated value of the land (the actual cost some years before was $45 000)..	250 000
Working capital......................	500 000
Total valuation..............	$1 850 000

* San Juan, Porto Rico.

Mr. Gandolfo.

The capitalization of this plant was:

Bonds	$3 000 000
Preferred stock	1 500 000
Common stock	1 500 000
Total capitalization	$6 000 000

(In this particular investigation, the present value of the plant would have had no possible significance.)

What would be thought of the private individual who, for every dollar that he wished to invest in a plant, issued obligations to the amount of nearly $3.25, obligations that constitute a lien against his plant, and that must be met and settled some day?

If a business man invests $1 000 000 in a private business, that is the value of his plant. If he wishes to raise money on it in any way, that is the figure which would be used, provided depreciation and obsolescence had not taken place. In the past, however, it has been the general idea that a corporation, whether a private or public utility, could issue securities of two, three, or four times the amount of the actual investment. Such securities, whether legally or not, are at least morally and ethically a "mortgage" on the plant. It is on these securities that the public must pay interest. It is such securities that constitute a burden on future generations, for ultimately payment or default is a certainty. This over-capitalization is one reason for so many public utility corporations being so insistent on the use of the present-value theory of appraisal, usually without any allowance for depreciation, because, on account of the generally increasing cost to reproduce such plants (as the writer has already pointed out), this theory gives a larger tangible value behind such securities.

The argument is often advanced that a corporation has no soul, that it is simply a great machine, grinding along irrespective of any one or any thing. Only a few months ago the manager of a large engineering corporation, which has built and controls many public utilities of all kinds, advanced this very argument to the writer. But, stop a moment, and briefly analyze the corporation. Men are responsible for its very existence. Men are responsible for all its acts and all its results. In other words, a corporation without the human element is a dead and useless thing. It is only the "bare bones" of the organization, without muscle, sinew, or brain. The human element gives it life, makes it a "going concern", and therefore men are as responsible for its acts and there is as much responsibility, morally and financially, resting on those who direct its smallest acts, as if these same people were engaged in private business.

Those who contend that a physical valuation for the purposes of rate-making or security issues should be founded on present value,

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with or without depreciation, or on replacement value, lose sight of the origin and *raison d'être* of both security issues and rates. They also fail to have analyzed the subject correctly, or to have investigated the anomalies and injustices that arise when any one of these systems is followed. In order to show this, the advocates of such systems are asked two questions:

(1) What are security issues for?

(2) What is a rate of return, as fixed by a commission or other controlling power, for?

Taking up the first question: The issue of legitimate securities is for the purpose of obtaining, by their sale, funds to invest in some enterprise, or to cover the value of money and labor already invested. Any other issue of securities, not backed up by good and sufficient assets (and included in assets should be such items as promotion fees, organization expenses, etc.), amounts to adding "water" to an otherwise legitimate enterprise.

Answering the second question: A rate of return is supposed to be such an income as will give a fair percentage of return on the capital invested in the enterprise. Such rate of return, of course, should be higher than the prevailing rate of interest, otherwise there would be no "profit" in the undertaking. The present or replacement value does not give the capital invested in an enterprise, and therefore a rate of return founded on such a value does not give a return on the capital invested.

To show how the present or replacement value, as a basis for the issue of securities, or as a basis of rate-making, may work great injustice to innocent investors, or to the rate-paying public, will be illustrated by the following example: The cost of the steelwork for three large power-houses, with the design and construction of which the writer is thoroughly familiar, is used as an illustration, as here is a standard which is probably affected less by unforeseen conditions and other contingencies than any other. For example, foundations differ widely in cost, due to sub-surface conditions, superstructions due to architectural treatment, etc. The construction of all three of these power-houses was similar, the main features being boiler-rooms and engine-rooms separated by a division wall, over-head coal bunkers supported on the steel columns, conveyor runways, monitors, etc.

These three power-houses will be designated as A, B, and C, and the year of erection of the steelwork, and the costs, were as follows:

Power-house.	Year erected.	Cost of steel per pound, erected.
A	1902	3.74 cents.
B	1906	4.07 "
C	1909	2.87 "

Now, suppose that a physical valuation of the steelwork for A was made at the time B was built, and that present or replacement value was used, and, in order not to complicate the problem unnecessarily, that no allowance for depreciation was made. This valuation would raise the value of the steelwork for A 8¾ per cent. If additional securities were to be issued to cover this increased value of the plant, the first question that would naturally arise is: To whom should they be issued? If they are to be issued at all, they must be issued *pro rata* to the security owners of record on the day such new securities were issued. Mr. Gandolfo.

Now comes the ethical and practical question as to what is to be done with the rate of return for A, assuming that the rate has already been fixed by law, on the basis of the original value, and that perhaps such rate has been tested and finally decided by the Courts. Should it be suddenly raised to pay a "fair return" on this new valuation, or should it be left at the old rate, which in percentage would now show a less than "fair return"? The utility has made no increase or betterment in service. It has invested no new capital. It has done nothing to benefit or help the rate-payer. This increase in value amounts to an unearned increment due to extraneous conditions over which no one has any control. If the utility is really the agent of the State, should not this increase in value belong to the principal, not to the agent?

Now, assume that a physical valuation of the steelwork for both power-houses, A and B, was made at the time C was built, and that present value was used. Under these conditions the value of the steelwork for A would decrease more than 23%, and that for B more than 29 per cent. Following out the same line of reasoning, it now becomes necessary to reduce the capitalization of both A and B, in order to conform to the present value as exemplified in C; but who is to give up these securities? The stockholders of record of A and B in 1909 may be and probably are an entirely different set of people from those of 1906 and 1902. Those who paid 3.74 cents in 1902, 4.07 cents in 1906, and those who further received the bonus on A in 1906, may have absolutely no interest in A, B, or C in 1909. To go to innocent parties, and say that they must give up arbitrarily part of their investment simply on account of a falling market for the materials of construction, amounts to the worst form of confiscation of property; and yet this is what the advocates of the present-value, or replacement-cost, or replacement-cost-less-depreciation theories of physical valuation, ask us to do, if the cost of the utility happens to have decreased.

Then here again, as before described, comes in the question of rate of return. Should rates be immediately reduced, to give a fair return on what is now the "fair value" of the plants? Would such

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a procedure be fair or just to the utility, any more than the former case would be fair to the rate-payers?

One of the principal objections to the actual-cost method of appraisal is that it deprives some one of the natural increase in value of the utility. In addition to those reasons given in the paper against the capitalization of such increase, is the following:

It is now claimed by many, and seems to be becoming more and more of an accepted fact, that the public utility is in reality only the agent of the State, and acts as such. If this is the case, then assuredly such increase in value belongs to the principal, not to the agent, except such percentage as might be agreed on as reverting to the agent as a part of his remuneration.

Again, who is the actual owner of a public utility? In many cases the records of the New York Stock Exchange show that the entire capital stock of some corporations has changed hands within a comparatively few days. Bonds and gilt-edged investment securities are also constantly changing hands. Unless there is some stable figure on which to base the value of these, investments will be in a constant state of uncertainty. The figure founded on present value is unstable, and is constantly changing.

The writer has also heard the argument advanced that, in a large utility, although there are many elements that have increased in value since it was built, there are also many that have decreased greatly in actual cost, and thus the two sets of elements tend to neutralize each other. If this were true, or even only approximately true, there would be no need of further argument and contention between the two schools of appraisal. Each would arrive at the same result by different methods, and all differences of opinion would be at an end.

Several of those discussing the paper have stated in their arguments that if the actual-cost-to-date method of appraisal is used, an expert accountant is all that is necessary to do the work, and that the trained engineer can be entirely eliminated. Such statements are without foundation in fact. An expert accountant is absolutely without the necessary training or experience along engineering lines to make any kind of an appraisal or valuation of a public utility property, no matter what system is used.

To begin with, the expert accountant is without any knowledge as to depreciation in any of its forms. Therefore, it is impossible for him to take into account what may be a very important factor, and possibly the most important one, in an appraisal. For example, the writer is at present in a position to take careful note of the condition of the rolling stock of a trolley line. There is not a single car on the entire line in first-class condition. It is a marvel how

some of them hold together. What would the expert accountant's book investigation show of such a condition? His figures would be the same, whether the plant had 100% in efficiency and condition, or was a scrap heap. **Mr. Gandolfo.**

Also, how can the expert accountant know, in many cases, what items should be charged to maintenance, and what to new construction? Or what items on his list of costs have been totally consumed, and have been replaced again and again? Such questions often tax the training and experience of the expert engineer to the utmost, in order to arrive at a just decision.

Secondly, such an investigator would be absolutely incapable of judging whether or not there had been unnecessary extravagance in the design and construction of a public utility property. The writer is familiar with a public utility power-house the superstructure of which was designed by architects totally unfamiliar with this class of work. An expensive flaring base was provided for all walls, entailing also much wider and heavier foundations than would have been necessary otherwise. There was also an expensive overhanging brick cornice, with terra cotta coping, surmounted with a heavy wrought-iron pipe rail. The main entrance vestibule was finished in marble, with marble seats of Roman design along the walls. These are only some of the totally unnecessary architectural features embodied in the design of this power-house which helped to place an unjust burden on the traveling public.

In another public utility power-house, among other totally unnecessary architectural and ornamental features, the turbine-room was finished in expensive tiled brick of different colors, extending from the floor to the crane rail. The crane rail itself was completely hidden by a projecting cornice of the same material. How would an expert accountant be able to judge as to whether or not such features, and others perhaps not so glaringly extravagant, were necessary to the successful and satisfactory, as well as economical, operation of the utility?

Finally, how could the expert accountant know whether or not there had been fraud in the entire development and construction of the utility? He could detect straight falsifying of the figures, if this had been resorted to; but such crude methods are not generally used. He would not be able to judge whether unit costs had been unduly increased, whether higher wages had been placed on the books than had actually been paid, whether padded pay-rolls had been certified to; whether unnecessarily high prices had been paid for materials and workmanship; or to detect the thousand and one tricks resorted to by unscrupulous promoters, who care nothing for the ultimate success or failure of an enterprise, but rely solely on the excess cost of construction, or other questionable methods, to reimburse themselves.

Mr. Gandolfo. By these facts it is easily seen that the expert accountant is totally incapable of making a physical valuation of a public utility property, no matter what theory of valuation is used. Those who advance such an argument show one of two things: Either they are ignorant of many of the necessary requirements of a physical valuation, or they are trying to support an untenable position by resort to arguments which are palpably weak and without foundation.

Several of those who have discussed the paper lay great stress on decisions of the Courts in regard to "value". The term "value" has never been defined. It is simply a matter of the opinion of one man, or of a set of men, and is often influenced by ignorance, prejudice, or bias. It has been said that value is what a thing will bring in the open market. There is no open market for a public utility property, and, on analysis, this test also fails in regard to other things. For example, one man will pay, perhaps, $40 000 for a single vase which another man would shatter with a blow of his hammer as being a useless incumbrance. As an example of different opinions as to "value", not only of intangible things, but of concrete physical work, no better case can be cited than that of a recent appraisal of the Chicago elevated railroads. On April 30th, 1912, George F. Swain, Past-President, Am. Soc. C. E., submitted his report on the value of the Chicago elevated railways, giving $34 634 396 as the depreciated value, exclusive of real estate, rights of way, and overhead charges. In this amount most of the uncertain items in a railroad valuation are omitted. For the same property, A. L. Drum and Company estimated $40 750 892, and the Harbor and Subway Commission of Chicago $26 354 217.

As to judicial decisions, these, too, change with time and circumstances. Furthermore, a Court must decide on the evidence as presented before it. It is the prerogative of the engineer to conclude what is just and right, and then present the matter before the Court so that no alternative is left but to render a decision along just lines. Very often it is not possible to leave this to the attorneys in the case, but the expert engineer must take the lead, in the preparation and presentation of the case, as well as in the mere physical examination and investigation. The writer once heard a late noted lawyer bungle his presentation of a case in Court—in which appraisal and expert opinion played the most important part—so as to astonish all connected with the proceedings. This is what engineers must guard against, in all valuation and appraisal cases with which they have anything to do.

The writer has great respect for the judiciary, and high regard and opinion for the Supreme Court of the United States and its decisions; but all Courts are composed of men, and no man is infallible, and the following instances go to show that even the decisions of this

great tribunal may become only matters of history. It is only a short hundred years ago that flogging was practiced in the United States Navy, and was upheld by law. What an outcry would occur if such a thing was attempted now. In 1857 the Supreme Court handed down the famous "Dred Scott Decision". This decision was accepted by not more than half the people of the United States. Mr. Gandolfo.

As already referred to in the paper, in the decisions relating to the valuation of public utilities, the Courts have several times said that the market value of the securities of a corporation should be given consideration in an appraisal. Does any one who is at all familiar with the fluctuations of securities, and the methods of manipulation as used in the stock markets, suppose that the market value of these necessarily has any possible relation to the physical value of the utility, even though so stated by the Supreme Court of the United States?

As examples of the fluctuations in securities, the following are given. In 1902 the stock of the New York, New Haven, and Hartford Railroad sold at \$255 per share. To-day it is selling around \$52, a difference of \$203 in 12 years. In 1913 it varied from $\$129\frac{7}{8}$ to $\$65\frac{5}{8}$ per share. The stock of the St. Louis and San Francisco Railroad in 1913 sold for \$59 per share. In 1914 it sold for $\$3\frac{1}{2}$ per share. The stock of the New York Central and Hudson River Railroad, one of the leading roads of the country, sold for $\$174\frac{1}{2}$ in 1902. To-day it is selling for \$83. Even such a stock as that of the Pennsylvania Railroad sold for \$170 per share in 1902, and this year is selling around \$105. If space permitted, a long list of both stocks and bonds could be given, with fluctuations even worse than some of those just quoted. This shows the utter fallacy of basing any conclusion on this element. Indeed, it is valueless in an appraisal, even for purposes of comparison.

Mr. Green and Mr. Mayer have hinted that the paper is a socialistic production. Particularly is this so with Mr. Mayer, who refers to the subject again and again in his discussion. The writer wishes to deny emphatically any idea of advancing in any way any socialistic doctrines whatever, or having any leanings whatever toward any of the socialistic propaganda. The paper sets forth some hitherto unpublished arguments and reasons as to why certain theories, already held by many eminent engineers and jurists, are just and reasonable. It is in no sense socialistic.

These two gentlemen have fallen into a mode of procedure that, unfortunately, is quite common to-day, namely, when any one advances arguments which do not happen to coincide with his own, or do not happen to suit certain preconceived notions of some particular section of the community, to try and disparage them, and also

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their author, by classing them as a socialistic propaganda, particularly if such arguments are otherwise difficult or impossible to refute. Such methods are without justification, and have no place in a scientific discussion before a scientific society.

From the opening paragraphs of Mr. Dow's discussion it is very evident that he does not appreciate the real question at issue; neither does he seem to consider that, in regard to appraisal and valuation, when the question of rates or security issues is at stake, there are two distinct schools of engineers at the present time, one holding that the actual cost of the property of the public utility is the proper figure to be used, and the other that the present value of the property is the one to be used. Mr. Dow's statement of the problem is not sufficient. A better statement is as follows: Wanted an appraisal for the purpose of determining rates, or to determine the amount of securities to be issued. Shall actual cost to date be used, or shall present value be used? The final answer has not yet been given. Also, the problem is not so simple as it appears from Mr. Dow's point of view. If it was, it would be unnecessary to discuss the matter before this Society, as he states when he says "A valuation is an engineer's work, and the manner of making it is properly a subject of discussion by this Society." What, under his premises, is there to discuss?

There is no confusion whatever in the writer's mind between "cost" and "value." This is all so clearly indicated in the paper, particularly on page 874, under Heading 6, as to need no other comment. Mr. Dow must have overlooked this part of the paper. Cost is an actual stable figure, known and positive. The principal thing about value is that it is unstable, and no one knows precisely what it is, as has already been shown.

Mr. Dow thinks that an appraisal or valuation based on the actual cost to date could be prepared by a competent accountant. The writer has already shown the fallacy of such beliefs.

On page 879 Mr. Dow quotes a sentence from the paper, and comments on it. This sentence is found in the early part of the paper, and unless the greater part of the paragraph in which it occurs is quoted, it has no particular force or meaning. It is not an argument, is not intended as one, and is simply an opening sentence leading up to a discussion.

On page 879 Mr. Dow says:

"Are we not setting up a false standard permitting that an engineer's opinion of value shall be different when his client proposes to buy, from what it is when he proposes to sell? Is it not the duty of an engineer called on to make a valuation to say that he finds certain property the present value of which is thus and so?"

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Then, in the very next sentence, Mr. Dow contradicts himself, and admits that the "engineer's opinion" of value may be different under different conditions, when he says:

"If the engineer's opinion is asked as to what would be a wise price to be accepted or offered for that property; or if he is appointed arbitrator to fix a price; or if he is asked to express an opinion as to a fair rate of return to be allowed on the property in a rate-making case—under any of these conditions, an opinion as to price or as to rate of return is proper; * * *."

All of this also directly contradicts Mr. Dow's own statement of the problem, as given on page 879.

Referring to the last two paragraphs of Mr. Dow's discussion—there is no false standard whatever set up for engineers by the actual-cost-to-date method of appraisal for rate-making and security issues; nor is there any necessity for any confusion in engineers' minds in regard to the question. The entire matter is one of logical reasoning and deduction. The question of appraisal is not one in which the engineer is to "inject his opinion", no matter what system is used, but is one in which he is to determine facts as he finds them. The only system that gives facts is the actual-cost-to-date system. The gist of the matter is that Mr. Dow has ignored the fact entirely that the public utility corporation is a creature of the State, and to a greater or less degree is the agent of the State. It must also be admitted that public utility property is held under a very different tenure from that of other property. Otherwise, the very fact of State regulation of rates, of security issues, and of the very details of the business, would be impossible.

Mr. Lavis, in opening his discussion, refers to the recent paper* by Mr. Alvord. This is not a discussion of Mr. Alvord's paper, and the writer regrets very much to have to refer to it, but, as it has been brought into this discussion, it is only necessary to say that the writer has read this paper, and finds that the same ideas seem to obtain therein as were set forth in a former paper by the same author, and were severely criticized by the writer.† The writer regrets that he has not had time to prepare a discussion of Mr. Alvord's second paper, setting forth in detail some criticisms of the statements embodied therein.

On page 880 Mr. Lavis says:

"It would appear at once that a valuation based on the actual cost to date method, * * * would by no means necessarily give a true estimate of the 'present-day value'."

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* "Fundamental Principles of Public Utility Valuation", *Transactions*, Am. Soc. C. E., Vol. LXXIX, p. 117.

† *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 863.

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On page 881 he practically repeats this statement. The writer does not know of any one who ever claimed that it necessarily did. It is strange how the advocates of the present-value method of appraisal persist in accusing the advocates of the actual-cost-to-date method of confusing the two results. The two results may be and probably are entirely different. No one makes any claim that they may be or ought to be the same. This is all clearly set forth in the paper. Perhaps Mr. Lavis overlooked it.

On page 881 Mr. Lavis says that it is often difficult to see why depreciation should be deducted from the reproduction cost of a machine that is giving 100% efficiency. If Mr. Lavis will refer to pages 869, 870, and 871, he will find cogent arguments there set forth as to why this question of depreciation should be given very careful consideration, and how it should be treated.

Furthermore, it is perfectly possible for a "machine" (using this term in its general sense, as well as specifically) to continue to give 100% efficiency, or perhaps very nearly 100% efficiency, up to nearly the end of its period of usefulness. The question of efficiency may have nothing whatever to do with depreciation, which is constantly going on, and much less with obsolescence and inadequacy. Of course, in a large utility, such as a railroad, different parts are constantly being replaced. At the same time, there is an example in the New York, New Haven and Hartford Railroad, showing how far depreciation and obsolescence (which is only another form of depreciation) can be allowed to go, and still permit the utility to give 100% efficiency up to the time that the inevitable collapse occurred, which was bound to come, under the conditions of deferred maintenance which existed on that road.

The rolling stock of the trolley company previously referred to is another example as to why depreciation must be considered in any appraisal. As far as transporting passengers is concerned, the old ramshackle cars are giving 100% efficiency, just as new cars would do, but sooner or later (and probably sooner) these cars must be replaced. They are not worth 100%, or anywhere near it, and it would be misrepresentation so to consider them, either in an actual-cost-to-date or present-value method of appraisal. The question of "efficiency" has no place in a purely physical valuation. When intangible elements, such as "going concern" are considered, then it is time to take up "efficiency", but in a different sense from that in which Mr. Lavis uses the term.

On page 881, Mr. Lavis says:

"* * * some general method must be adopted which will be equally applicable and fair to all the railways of the country, to determine with a reasonable degree of accuracy and fairness their present-day value, * * *."

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This is a bare statement of Mr. Lavis' opinion, without any supporting arguments or facts, and he is simply "begging the question" when he uses the term "their present-day value". This is the very thing that is under discussion, and Mr. Lavis has advanced no arguments to show why "actual-cost-to-date" is not just as fair, and should not be used in place of his words, as just quoted. Nor has he refuted any of the arguments of the writer showing why this method should be used.

Mr. Harte, in his opening paragraph, attempts to deny that there are very material differences, in some respects, between the public utility corporation and the private business. This is not substantiated by the facts. These differences are all set forth in the paper on pages 852, 853, 854, and 878, and at a glance it can be seen so clearly, from the very fact of detailed control of the public utility, its property, its acts, and the conduct of its business by the public utility commission of the State, or other controlling body, and it is also so well testified to by the various Court decisions in every line relating to public utilities, as to seem to need no further comment.

Referring to the second paragraph of Mr. Harte's discussion, if he will turn to page 848 he will find the following sentence: "Any business is susceptible of endless modifications and extensions, * * *." This he evidently ignores. "Wildcatting" is just as possible in a private business, and is just as reprehensible, as in any other, but there is this great exception, compared with the corporation: Such business methods cannot be hidden behind securities, as they can be in the stock company, and innocent people brought into the matter without their knowledge and consent.

Mr. Harte is absolutely mistaken when he says, on page 882, "Proportionately, the private business deals as extensively with that portion of the public in its field as either of the other groups; * * *." Why the private business does not, and cannot, is so well set forth in the paper as not to require repetition or further explanation; also, his statement that the "liability is proportional to the holding", is not borne out by the facts. Everything the owner of a private business has is behind that business. If he has invested $100 000 in a private business, and his personal assets are $1 000 000, all of this can be levied on in case of failure; but if he held $100 000 of securities in a company, this would be all that he would be liable for, no matter what his personal fortune was. Mr. Harte admits this fact on page 883 when he says:

"* * * modern business demands such large capital that few care to put so large a part of their money in one investment, but would rather divide it among many projects, in order to reduce the extent of the loss in case of failure of one or more."

All of this was fully set forth in detail by the writer on page 849

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Mr. Harte's third paragraph is not very clear, but the writer supposes he meant to say that if an old concern wished to expand and build additions, and estimated the cost of such additions as the same as the original cost of the old plant, instead of basing it on the present value of labor and materials, that they would very likely fail. Of course, they probably would. If any one did anything as foolish as that, he would deserve to fail; but what this has to do with the subject, or what point in physical valuation it elucidates, is not easy to see.

On page 882 Mr. Harte refers to the use of the roads by teams. He might as well say that if a man calls a policeman to arrest a thief, he is asking a favor of the State, or of the municipality. He is not; and neither is the teamster or firm that uses the road. In cases of this kind, the citizen is exercising his right, for which he pays in taxes, and is asking no special favor of the State. Marquee and areaway permits, etc., also mentioned in this paragraph, are not special privileges, but are such as any citizen, in like circumstances, may obtain. Applications for these by individuals are not in any way connected with the special exclusive favors from the State under which a public utility corporation does business.

The writer wishes to assure Mr. Harte that he is thoroughly familiar with the law of eminent domain, and knows exactly why it was necessary to enact such a measure. The writer did not say it was a special privilege to further the affairs of a corporation. What he did say on page 848, in referring to the private business, was, "Nor can it invoke the right of the law of eminent domain to further its own affairs".

On page 883 Mr. Harte says that a partnership is nothing more than a corporation of special form. This is not true in the accepted meaning of the words "partnership" and "corporation", both generally and legally. The only sense in which it can be said to be true is in an economic one, and this is not the way in which Mr. Harte used these terms.

On page 883 Mr. Harte states that the majority of the security owners of a utility control it. This is only so theoretically, and it is by no means so practically. When the securities of a public utility are widely scattered among perhaps thousands of stockholders, it is a physical impossibility for many of them to attend meetings, even if inclined to do so. Therefore, it is absolutely impossible to secure the attendance of anything like a majority, or even a very large percentage of stockholders at any such meeting. Proxies are obtained by the board of direction or other managers of the utility, and thus the entire control of the business is often kept entirely in their hands for years. Does Mr. Harte think that a majority of the stockholders of

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the New York, New Haven and Hartford Railroad agreed to the business methods that led to its undoing? Or that a majority of the stockholders agreed and gave their sanction to the methods of conducting business as followed by the 'Frisco Lines, or the Rock Island Lines?

When Mr. Harte says, on page 883, that Principles 2, 3, and 4, as given on page 852, are as characteristic of private business as of corporations, he fails to consider the difference between the methods of borrowing money, generally on notes, followed by private business concerns, and the method pursued in lending money on securities as collateral, which latter may not mean, and generally does not, that the concern whose securities are borrowed on is borrowing the money at all, but some outsider is raising the money for something entirely different. This matter of raising money on securities could be gone into at great length, but a glance at the entire matter is sufficient to show the radical difference between this and the borrowing by a private concern.

The writer fails to see how the Sherman Act and the cases decided under it, mentioned on page 883, have anything to do with the question under discusion.

In the literature on the subject of physical valuation, Reasons 1 and 2, on page 856, are two of the three most often advanced by present-value advocates of appraisal. They were not gotten up by the writer purely for this paper, as the wording of Mr. Harte's remarks would seem to indicate. If they are not correct reasons, it is the fault of the present-value advocates and the judiciary.

In regard to Court decisions, the writer does not ignore or deny them, as is set forth on pages 854 and 856, but Court decisions do not prevent engineers from striving for the right, and doing all in their power to bring about just and equable solutions of questions that are of vital interest to the State. If Court decisions are the only ground on which present-value advocates can base their case, as Mr. Harte hints in discussing Reason 3, then, indeed, is their case weak. For, if a premise is ultimately found to be unjust, even though supported by Court rulings, it must ultimately be abandoned, and just and stable ones substituted.

As to the "general rule", the writer is familiar with this expression, but sees no reason to change any of the wording on page 858 in regard to it.

Another reason advanced by Mr. Harte for the present-value method of appraisal is that it warrants the professional existence of engineers. If this is one of his arguments, then, indeed, is his case still more weak. For it is one of the fundamental ethics of the Profession, if the engineer finds he is not needed, to say so, and not try to "make a case", as is so often done by a certain class in the legal profession. Be this

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The actual-cost-to-date method does not deprive enterprise of its reward, nor does it accept the "unwarranted expenditures of the dreamer", as is all set forth in full detail on pages 855, 856, 862, 872, and 873. Mr. Harte's remarks in the fourth paragraph on page 884 do not appear to the writer to be consistent with the facts.

On page 884 Mr. Harte quotes from the Minnesota Rate Cases.* Referring to this case, the Court refused to allow the reproduction cost of real estate, as is particularly set forth by Justice Hughes on pages 44, 45, and 46.* Furthermore, on page 45* Justice Hughes says:

"The cost of reproduction method is of service in ascertaining the present value of the plant, when it is reasonably applied and when the cost of reproducing the property may be ascertained with a proper degree of certainty. But it does not justify the acceptance of results which depend upon mere conjecture."

Now, if it is wrong to appraise or value the land of a public utility at its reproduction cost (as Justice Peckham also hinted in Willcox *v.* Consolidated Gas Co.), is it not wrong to value any of the property at reproduction cost? As far as this is concerned, is not this decision contradictory? How can one rule apply to one kind of property, and another rule to another kind? It seems as if the Supreme Court began to realize the injustice of the reproduction-cost method of appraisal, but on account of the "general rule" was loath to say so.

Of course, the writer, in an actual appraisal, does not advocate basing any figure on "an old print". This, of course, would be absurd. The "old print" was simply used as an illustration in an argument; but, very often such things are of great value in an investigation.

In regard to trestling, the writer uses the term "temporary trestlework" and to this extent Mr. Harte has misunderstood him. Of course if a permanent trestle has been replaced at a higher cost, this is the figure that should be used. This is covered by Item 7 on page 855, which Mr. Harte evidently overlooked. Therefore, his statement that, according to the writer's argument, such new material would be charged to operation, is not correct.

In regard to Mr. Harte's arguments about purchase and sale, these in no way change the fact that a public utility should be valued at actual cost to date for rates and security issues. To begin with, a public utility cannot sell or transfer its property without the express permission of the State. "* * * the property so used is charged with a public trust and is devoted to a public purpose. Such property is dedicated irrevocably to the performance of this trust due the public and for its benefit and that of the inhabitants of the municipality."

* Senate Doc. No. 54, 63d Cong., 1st Sess.

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(Pond.) A study of Court decisions shows this. Now, suppose that the actual cost of a utility has been $1 000 000, including all items, and that a rate of return of 8% has been allowed, giving a net yearly profit of $80 000. Now, suppose that the present value of the property is $1 500 000, or that it has increased 50% in value. This in no way affects the original cost. If a purchaser now comes along, and after permission to sell is obtained, he buys this plant at this figure, the income would still be 8% on $1 000 000, or 5⅓% on $1 500 000. The new owner has bought with the express knowledge of what he is going to get and why he is going to get it. The actual cost to date is not changed, the rate of return is not changed; and, being a public utility, and agents simply having been changed in a practical sense, there is no reason why the rate should change. Thus no injustice is done to any one interested in the transaction.

In the last paragraph of his discussion Mr. Harte speaks of a "free sale" of a public utility. There is no free sale whatever for public utility property and never can be. The fact of there being no free sale, either from a market standpoint, or from a legal standpoint, for such property, is one of the facts that many present-value advocates seem to wish to ignore.

Mr. Green says that the principles of valuation must satisfy the requirements of law, of economics, and of engineering. This is true; but it must be remembered that two of these sciences are not positive, and the human element enters very strongly into them. The only one of the three that is positive is engineering, and this one only when all conditions relating to the particular problem under advisement are absolutely known. As far as appraisal and valuation are concerned, this branch of engineering is less positive than any other, and partakes more of the nature of speculation and surmise, unless the actual-cost-to-date method is used. Then all unknown elements are eliminated to a greater degree than by any other method.

As for principles of law, these are mostly man-made (as Mr. Green evidently refers to common law and statute law, and not to Nature's laws), and are constantly changing. Law is supposed to embody principles of right and wrong, but even these are purely relative terms. The code of ethics of the head hunter of the Philippines is entirely different from that of the people of the United States, and his customs and laws are, therefore, entirely different. Many ancient and medieval laws are now looked upon by us as having been harsh and cruel, and yet the people of those days thought nothing of them. According to his code of ethics, the engineer must keep constantly in mind what is right and wrong, and strive in every way to advance the right.

The writer wishes to assure Mr. Green, as well as he has Mr. Harte, that he thoroughly understands the law of eminent domain, and just

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why it was necessary to evolve it. The fact remains that the private individual cannot resort to this law, to further his own affairs. If resort is had to this law, a public necessity must be shown. The power of the law is not delegated to any one. Only the State can exercise it. If the law is to be involved, recourse must be had to the Courts. The public utility or "carrier" cannot go to any one who stands in its way and say we want your land, or your buildings, or your property, and simply take them. This is what the remarks on this subject, as Mr. Green puts them, would seem to indicate could be done. Condemnation proceedings can only be carried on through the Courts.

The writer fails to see what connection there is between one road going to the Courts to condemn a right of way across another road and the method to be used in a valuation.

Referring to Mr. Green's remarks on watered stock, this may be "faith", but is more apt to be "fraud". As for the farmer who asks $20 000 for his land, he does not and cannot ask the public to pay an income on his "watered value"; but, when watered stock is issued, it is expected that dividends will be paid thereon, and it has already been shown how such stock becomes a burden on the public.

Mr. Green does not realize the function of government when he speaks of fools and their folly on page 888. The fact is that a large part of any government is organized for, and occupied with, the express work "to protect fools from their folly", or, to put it in different words, to protect honest people from the pitfalls and machinations of the dishonest elements of the population. It is perfectly true that all governments, in undertaking this work, "have a bigger job than the Panama Canal".

Bread, clothing, and recreation are certainly vital to a community. But a bakery, a haberdashery, or a picture show is not. If any one of these is not satisfactory, or if satisfactory service is not given, there are hundreds of others where the consumer can go and get satisfactory service. If one is destroyed, or fails, or goes out of business, the community at large is not affected in the least. The patrons of this one distribute themselves among others of like kind.

With a public utility, however, this is impossible. As it is a monopoly, the public must patronize the utility or go without. Therefore, Mr. Green's criticism of the writer's definition is apparently not well founded.

It is not a question of one law for one enterprise and a different law for another kind of enterprise. It is a question of having a just law, impartially enforced, applying to all enterprises that exist under certain conditions and circumstances.

The writer has carefully examined pages 854 and 855, and fails to find any contradictory statements whatever thereon. One sentence does not make a book, or an argument.

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To show why a valuation cannot and should not necessarily be the same for all purposes, no better example can be given than that of the recent appraisal, by the Comptroller of New York State, of the Hales Bar development on the Tennessee River. Here was a development which had cost more than $10 000 000; which should not, if reproduced, cost more than $5 000 000; and which was in no sense a going concern, and with no immediate prospect of being one. To appraise this development at actual cost, for the purpose of the inheritance tax, would certainly not be fair to the owners. To appraise it at present value would be meaningless, because, strictly speaking, as the plant stands, it has little more than scrap value; but, as the heirs signified their intention of carrying on the development, this plant certainly had some value for purposes of taxation, as was finally decided when its inheritance tax value was fixed at $3 200 000.

As to a valuation for taxation, it has already been shown on page 876, that it makes no difference to the utility or the rate-paying public whether such a valuation is high or low. The entire matter is not a question of low or high valuation, as Mr. Green puts it, but is one of a just and equitable appraisal and valuation to all concerned.

As far as the acquirement of land goes, it is an impossibility for any public utility to look far enough into the future to meet all its requirements. Furthermore, granting that it could, what is the limiting period in the future for which present generations may be taxed in order to provide for future generations' needs? How much unproductive investment is a public utility justified in making, for future needs and developments, and how much is it justified in asking present generations to contribute toward such investment? Very little, if Court rulings are an indication of anything in this line.

The writer does not criticize roads which are now improving their lines and grades or making other betterments, as Mr. Green says he does on page 865. Referring to this page, it will be found that the writer says such changes must be paid for out of profits, if they take the place of old work. This same matter of additions and improvements is also covered by Items 7 and 14, on pages 855 and 856. Mr. Green, in the same paragraph, links the question of excess size of plant with this subject. What possible connection there is between the two, the writer is unable to see. Betterments and improvements certainly have no connection with excess size of plant. This question of excess size of plant has already been covered in discussing the future requirements as to land, and the same arguments apply to the entire plant. Therefore, there is no meaning in the last two sentences of Mr. Green's discussion, as applying to the principles of appraisal and valuation as advanced by the writer.

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At the very beginning of, and all through, his discussion, Mr. Mayer persistently says the writer's ideas are socialistic. The writer has already referred to this matter, has disclaimed any leaning toward socialistic doctrines, and wishes again to emphasize this fact.

In his first paragraph, Mr. Mayer says, "The idea that compensation should be in proportion to the effort made to secure it, is the leading motive of his paper, * * *." However, there is nothing in the paper to indicate this. If Mr. Mayer will refer to pages 855 and 856 he will there find fourteen items which cover all things to be taken care of in a physical valuation. There is nothing socialistic here. On pages 860 and 861, in discussing an enterprise that has failed, there is absolutely nothing socialistic in the treatment of this subject. Again, on page 862, in discussing promoters' fees, there is nothing of a socialistic nature, and on page 873, where the writer says a return should be allowed over and above the ordinary rate of interest return, there is certainly nothing partaking of socialism.

A public utility is not merely the tool of a private party for obtaining a revenue. It is primarily a tool to serve the public, and must be run and managed with this end in view. It thus differs widely in this respect from the private business.

On page 889 Mr. Mayer says: "Therefore, those who attempt valuations of complicated enterprises by ascertaining their cost of production meet with various unsolvable problems, * * *"; but he neglects to give any list of such unsolvable problems.

On pages 889 and 890 Mr. Mayer goes into a lengthy argument on the subject of competitive enterprises, and attempts to explain at length some matters connected with such enterprises. Mr. Mayer does not appear to realize, that with the vast majority of such enterprises there is no competition whatever; all arguments and reasoning in relation to them based on any such competitive theories are founded on false premises, and are, therefore, fallacious. This is the principal point in the entire matter, as stated by the writer at the top of page 854; and any one who does not understand and admit it, is simply groping in the dark.

In the last paragraph on page 890 Mr. Mayer says a mere percentage on the cost of an enterprise cannot be said to be any criterion as to the compensation for promoters and bankers. This is exactly what the writer says in the last paragraph on page 862, and shows on this account why such items should be put in at "cost", not at a "fair value".

On page 891 Mr. Mayer says: "Why not guess the total value without going to the trouble of ascertaining first the physical value?" This remark is so ridiculous, on the face of it, as to need no further comment.

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On pages 891, 892, 893, and 895, Mr. Mayer refers to what he calls a "guessed at" going value, and says the writer advocates this. On page 891 he is talking about competitive enterprises, and says: "The author thinks that an arbitrarily chosen or guessed at going value should be added to the physical value in order to obtain the total value of an enterprise." The writer did not say anything of the kind in relation to a competitive enterprise, as a reference to pages 846, 847, and 872 will show. The word "guess" in this connection has been created by Mr. Mayer. In the case of a public utility, where the question of rates is to be determined, will Mr. Mayer please inform the writer, and the Engineering Profession at large, how else the going value can be determined except by a more or less arbitrarily chosen figure, founded on a careful study and investigation of the particular problem under consideration? This, the only method there is, is in no sense a "guess", in the meaning of this term as used by Mr. Mayer.

In discussing this question on pages 892 and 893, Mr. Mayer uses such terms as "* * * same profits as are obtained in competitive enterprises * * *", "fair market value", and "competitive profits". As has already been pointed out, none of these things exists in a public utility which is controlled by the State.

Now assume, as Mr. Mayer does, that a public utility has its rate fixed. This gives the future net earnings with a reasonable degree of accuracy. Then suppose these net earnings are taken as the going value (not being any competition, there can be no competitive earnings) and are added to the physical value to obtain the present or fair value. Then rates must be allowed on this new "fair value", and so on, *ad infinitum*. The fact is, that every one, the Courts included, after a careful study and investigation of each case, has been compelled to arrive at going value for a public utility by what Mr. Mayer has been pleased to call a "guess", when the question of rates has been under advisement. A study of legal decisions shows this. (Knoxville *v.* Water Co., 212 U. S., 1; Cedar Rapids Water Co. *v.* City of Cedar Rapids, 118 Iowa, 234, 91 N. W., 1081; Cedar Rapids Gaslight Co. *v.* City of Cedar Rapids, 223 U. S., 655; Cumberland Tel. and Tel. Co. *v.* City of Louisville, 187 Fed., 637; and many others.) To say that the revenue is to be capitalized, when the revenue is the very thing to be determined, is "begging the question", and is a farce on the face of it. Mr. Mayer's reasoning along these lines is fallacious.

On page 892 Mr. Mayer says: "He also claims to show that the same principles should be applied to the valuation of the property of a private individual", and then mentions a buyer and seller. Just what "principles" he refers to here is not clear. Furthermore, the writer was not applying any course of deductions to a private business, but was showing the development of the public utility corporation from the

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private business. As for the "sane" buyer and seller, all this is set forth on pages 846 and 847 exactly as Mr. Mayer puts the theory, only very much more in detail. The writer feels sure that Mr. Mayer must have entirely overlooked these two pages.

The writer begs to tell Mr. Mayer that he is not "obsessed" by the "theory" that intelligence deserves no consideration in a valuation, as a glance at pages 855, 862, 872, and 873 will show. The writer supposes Mr. Mayer overlooked these pages.

The writer did not go into any extended detail as to how the actual cost to date of a public utility is to be obtained, for the very simple reason that his paper is an argument as to why actual cost to date is to be used, not how it is to be obtained.

In the third paragraph on page 893, Mr. Mayer says the writer's notion as to how profits are obtained in a competitive business appears "preposterous". Then he goes on to show how such profits are secured, all of which is exactly as the writer puts the matter on pages 846 and 847, particularly the first paragraph on page 846, only much more in detail. In other words, Mr. Mayer says the writer's "notion" in regard to this is "preposterous", and then goes on to give, as his own ideas, exactly what the writer has already said. This is somewhat peculiar. On page 872 the writer tells in a general way how a profit is figured in a competitive business. Mr. Mayer must know that in nearly every business, especially a manufacturing business, which corresponds more nearly to that of a public utility business, careful detailed costs are kept of every item entering into the finished product (both material and labor items), so that the actual cost of the product may be known, a profit made by adding a percentage to this cost, and the said cost of production watched and reduced wherever possible, for purposes of competition.

Another thing Mr. Mayer has entirely overlooked is the fact that on page 848 it is stated: "Any business is susceptible of endless modifications and extensions * * *", and again on page 877, "* * * so also every public utility is capable of endless modifications and variation * * *."

On page 893 Mr. Mayer says a State that engages in business is interested in the profits or losses resulting therefrom. A State is interested in the losses, but is not interested in the profits. A State goes into business to supply some particular thing to its citizens at large, not to make a profit. Further, he says here the cost to date must be ascertained. As the owners of a public utility are practically only the agents of the State, then the cost to date of the enterprise is the figure to be ascertained, on which to calculate the profits, the profit in such a case being the compensation of the agent.

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On page 894 Mr. Mayer says the writer's reasonings are quite indefinite, on account of lack of definitions for the terms used. The writer did not intend to use terms or words in any sense other than that of their generally accepted meaning, and believes they all are perfectly clear.

On pages 894-95 Mr. Mayer talks of "competitive profits", in relation to a franchise of a public utility. There is no such thing as competition in any form in connection with a public utility (except in a very few rare and exceptional cases that can be ignored), as has been shown several times. Mr. Mayer seems to be "obsessed" by this idea of competition in connection with public utilities. His remarks based on this are meaningless.

His elaborate formulas on page 895 have also been shown to be meaningless, so far as they possess any value for determining the value of a public utility when rates are to be determined. To capitalize the revenue, when the revenue is to be determined, has already been shown to be an impossibility and a farce.

Mr. Mayer says that he believes the paper, as a logical product, is without value (page 892); that its definitions are not clear, and that it does not give an accurate description of any method of valuation (page 895), and also states that criticism should be without reserve, when one succeeds in getting a hearing before a scientific society (page 895). The writer is willing to let his paper itself answer such criticisms.

The fact of the matter is, Mr. Mayer shows very conclusively that he has not read the paper carefully, even for the purpose of his own discussion, much less to have studied the lines of reasoning as therein followed to their logical conclusions.

Conclusions.—Before concluding this argument, the writer wishes to call attention to the fact that in two discussions he has been misquoted. The writer believes that misquotations are inexcusable, particularly in a scientific discussion.

In attempting to refute any line of argument, it is necessary to follow it step by step, showing if possible at every point wherein it is fallacious and then setting forth at the end wherein the conclusions are erroneous. It is then necessary for the discourser to give arguments in his turn showing why his premises are the correct ones, following this with his conclusions, founded on his premises. This none of those who have discussed this paper has done, or attempted to do. Each one has picked out an item here and there, and has attempted to show that it is not founded on sound principles of reason and justice, without giving any heed whatever to the main subject matter of the paper, and what preceded and what followed.

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The writer has taken up these arguments in a general way, and then has taken up each discussion in detail, and has shown wherein they in turn are not founded on sound principles of reasoning and justice. Such expressions as "* * * it seems only necessary to refer briefly to the matter in this discussion in order that Mr. Gandolfo's statements may not become part of its official records without some protest", quoted from the discussion by Mr. Lavis; "* * * one of the few rays of bright light in the fog in which so many writers have concealed the facts", from that of Mr. Harte; "Although the writer believes that the paper, as a logical product, is without value"; and "The author's notion as to how profits and values are ascertained * * * appears preposterous * * *", both taken from that by Mr. Mayer, are not arguments (and such remarks never can be), and do not in any way refute the statements and conclusions of the writer.

Therefore, in the absence of any logical arguments or reasons advanced in refutation of those in his paper, the writer must conclude that his premises are correct, his reasoning logical, and his conclusions just and true.

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TRANSACTIONS

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RIVERS AND RAILROADS IN THE UNITED STATES*

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With Discussion by Messrs. F. Lavis, H. Burgess, H. T. Pease, J. H. Bernhard, Emile Low, and William W. Harts.

Synopsis.

The large appropriations of public funds for the development of the channels in our rivers and harbors have grown from very modest beginnings.

Before the application of steam to transportation, the principal methods of communication were by horse-drawn vehicles or by sails. The invention of the light-draft river steamboat offered many facilities for the development of large areas otherwise inaccessible, and the demand for better channels in our interior rivers at once grew. Later, some of our harbors required better facilities, so that, from that day to the present, expenditures for these purposes have increased year by year until now there are projects under construction, or favorably considered, amounting to more than \$289 000 000, and the annual appropriations have increased to more than \$40 000 000.

On our rivers, the engineering methods adopted for accomplishing the work for which projects have been prepared involve all the best principles of river engineering in the world. They are surpassed nowhere in their excellence of design, suitability to the work for which

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proposed, economy of construction, and success in operation. On the economic side, however, there are new questions to be solved involving returns on the outlay. It is in this section of the subject that surprises are met, and many disappointments beset the river engineer.

In spite of the enormous sums of money expended on the rivers of the Mississippi Valley, there are but few that are holding their own as commerce carriers. The great majority are dwindling in traffic, so that it is apparent that their present condition, as to the depth and availability of their channels, is in advance of the use being made of them. Whenever streams do not show an increasing traffic which is at least approaching the increase of commerce in the region traversed, it shows that the usefulness of the stream in question is declining, and the necessity for continuing large expenditures is open to doubt.

There seem to be three fairly distinct stages in river history; the first covers the period commencing with the earliest times, the application of steam to vessels, and continuing up to the expansion of railroad building; the second stage covers the period from the rapid expansion of the railroads up to recent years; the third, in most sections of the United States, is still in the future. During the first period, commerce on the streams increased rapidly with the development of industrial conditions. During the second period, the railroads have taken over most of the river business, leaving the rivers struggling for a minor share of the traffic. During the third period, the prosperity and density of the population of the vicinity will afford sufficient traffic for railroads and rivers together.

In general, the United States, principally the central part, is now in the second stage, and the public is beginning to realize the necessity of curtailing expenditures until the time shall come when the industrial conditions will prove beyond question the need for using our rivers for special kinds of freight.

This disappointing record of interior streams does not apply to most of our harbors, and many streams on the coast are exceptions to the general conclusions mentioned; but there can be no doubt that, in the interior development of the nation, the usefulness of interior rivers as lines of communication has become much less important in late years. There are growing evidences that this is being perceived by the public, and we may soon look for restrictions in the large expendi-

tures now being made for all projects not showing a healthy and profitable growth in traffic.

In most of the progressive countries of the world, especially in those where population is dense and production large, the improvement of the waterways by the Government, for better communication in the interior, and the protection and deepening of harbors, have gone on at a rate which, as a general rule, has kept pace with the increase in population and production. Of late years, however, this progress has been strongly affected, both in America and in Europe, by the development of railways—sometimes favorably and sometimes otherwise.

In the United States the improvement of rivers by Federal appropriations extends far back into the early history of the nation, long before the era of railways. Before the first locomotive ran on rails and before the first steamboat had demonstrated its practicability, communication between different parts of the country was maintained chiefly by the sea, the interior being wholly dependent on roads and canals, and horse-drawn vehicles. Even during this time, while the interior was undeveloped and the population restricted to a comparatively narrow belt along the seacoast, these means were inadequate, and in proportion as the early pioneers conquered the unopened regions to the westward, the necessity for better communications became more and more urgent.

The Federal Government in those early days was not so compactly knit in its administrative processes as at present, and the improvement of interior waterways by Federal assistance was not admitted generally to be a proper Governmental function. These matters were largely left at that time to the initiative of the individual States; but the result of this policy was soon found to be so unsatisfactory and the road and canal work of the country so badly co-ordinated, that in 1824 all this improvement was placed under the charge of Army Engineers and given Federal support, in the hope that the early deficiencies might be met and obviated.

Work on these canals and highways required many engineers. Up to 1824 the Military Academy at West Point was the only school in the country which trained men as engineers, and for many years thereafter it was the principal school of this kind. It was only

natural, therefore, that the Government should have looked to its specially trained men to lay out and construct the work that was being done by the country at that time.

With the coming of the steamboat, the Government's interest in canal and highway work was soon expanded to include the improvement of rivers and the deepening and protection of harbors as the necessity therefor grew, so that, by a natural process of development, the duty of designing and constructing the enormous waterway projects which have since come into existence was the logical inheritance of the Army Engineer.

Until 1860 this work was confined mainly to the improvement of interior rivers, harbor work being still comparatively unimportant, although the project for the mouth of the Mississippi River, in all essential respects similar to what was constructed by the late J. B. Eads, F. Am. Soc. C. E., in 1874, was designed and proposed by an Army Engineer Board consisting of Colonels Barnard, Beauregard, and Howell, in 1854.

The light river steamboat jumped into favor at once as a popular method of communication, and showed very early that some improvement of the interior streams was plainly necessary. Many river cities, such as St. Louis, Memphis, and Cincinnati, trace their original importance as commercial centers to their favorable location on navigable streams. Up to this time, the increase in depth of ocean-going vessels had not had a very strong influence on the deepening of our harbors, for many ports having adequate channels could be found along our coast. Our rivers, therefore, received the earliest attention, and established first their claims to Federal aid. This work increased in volume and effectiveness up to the time of the Civil War, but all such work was abandoned during that conflict. In 1867, river improvement was again resumed, and the impetus received by all commercial business after the war was felt in this kind of work as well. At present the river and harbor projects, so modest in the beginning, have grown so that the work requires more than $40 000 000 annually.

The very early records of public money expended are so incomplete that they can scarcely be used to illustrate the enormous growth of these works, but between 1875, a comparatively recent date, and 1914, the annual appropriations increased from $5 218 000, in round numbers, in 1875, to $53 000 000 in the proposed Act of 1914, or more than

eightfold in about 40 years. These large sums are appropriated and expended in accordance with projects prepared by the Army Engineers and approved by Congress. The responsibility, therefore, must be shared between these two in some proportion, however unequal.

Very few people realize the amount of river and harbor improvement already adopted or under construction. Projects which have already received the sanction of Congress and are under process of construction now amount to more than $200 000 000. About $86 500 000 in new work has been reported favorably by the Engineer Department, but has not yet been adopted by Congress. In addition, there are projects amounting to more than $2 500 000, which have been favorably considered, but not yet accepted by the Engineer Department. Thus, there is a total of work costing more than $289 000 000, which is either in progress, or approved by Congress, or soon to be brought before the country.

Among the prominent works under construction, there are now 61 projects on the Great Lakes, the total estimated cost of which is more than $63 000 000; on the Atlantic Coast, 200 projects involving more than $100 000 000; on the Gulf Coast, 42 projects involving nearly $20 000 000; and on the Pacific Coast, including Alaska and Hawaii, 28 works requiring $22 000 000. The total of more than 300 different projects requires an expenditure of more than $205 000 000. It may be interesting to know that the work done on these projects up to June 30th, 1913, on the Great Lakes, has been carried on at an actual cost of 3.9% less than the estimates; on the Atlantic Coast, the actual cost has been 11.1% below the estimates; on the Gulf Coast, 8.7% below the estimates, and on the Pacific Coast, 8.8% above the estimates, the total for work done, up to June 30th, 1913, being 6.17% below the estimates.

It has been officially reported that in the 100 years, from 1802 up to December, 1902, there had been spent by the United States Government $221 869 759 for rivers, $147 448 903 for harbors, and $33 237 857 for canals, or a total of more than $400 000 000. From 1814 to 1900, France spent $449 000 000 on new works and in the maintenance of those already built. Similarly, Belgium spent $101 000 000 between 1831 and 1903. Between 1813 and 1906, Prussia spent $129 000 000 for new construction alone, and her maintenance charge in the single year, 1905, was $4 000 000. The United States is

about eighteen times as large in area as either France or Germany, and has about four times the length of navigable streams. Although the density of population in the United States is much less, being only about one-half that of all Europe, and about one-twelfth that of Germany, it can be seen that the United States has more than kept pace with other progressive nations, when density of population is considered.

As to the character of the work done by the United States in the improvement of waterways, reference may be made to the report of the National Waterways Commission of Congress sent abroad a few years ago to study waterways in Europe. In this report, it is stated that America had but little to learn from European works. This referred mainly to the excellence of the engineering features of our own waterways. A study of the methods and appliances in use in older countries convinced the Commission that there is at present no modern successful plan for the improvement of waterways which has not been better done in America. Furthermore, the engineering difficulties are of greater variety and magnitude and of more serious character here than in any country abroad.

To illustrate the high character of the work already accomplished, the Upper Mississippi may be pointed out as a model of open river regulation which, for this kind of improvement, is excelled nowhere else in the world. The canalization projects of the Great Kanawha and the Ohio, using movable dams, are models of their kind. The training of the mouth of the Mississippi River is an excellent and widely known example of the successful improvement of river mouths in tideless seas. The deepening of the harbor entrance at Galveston by jetties, the improvement of the entrance to the harbor of New York by deep dredging, and the construction of locks in St. Marys River, are all noteworthy examples of successful and highly developed engineering works which stand pre-eminent in their class. Even the system of storage reservoirs at the head of the Mississippi River is the largest and best managed of any in the world, and the system of levees on the Lower Mississippi, although not complete, has developed a standard and pointed the way for all future work of this kind. The country may thus take pride in the accomplishments of its engineers, and give deserved credit to them for initiative, courage, and skill. Excellent

engineering facilities for an enormous water-borne commerce have already been provided for, and are still being added to.

The first question that will be asked the river engineer is, to what extent are these facilities used, and do they pay in public service for the enormous cost of construction and maintenance? When we begin to study the economic features connected with works of this character, we are struck with surprises. The advent of the light-draft, inexpensive steamboat made our rivers and waterways of great value as commercial carriers at a very early date. These vessels, built with great propelling power and long, balanced rudders, carried enormous loads on very light displacements, often transporting several trainloads capacity of that period on hulls having a draft of less than 6 ft. The shallow and often swift rivers were responsible for this type, which was evolved after bitter experience. So successful did these vessels become soon after the Civil War that there were but few rivers in the Mississippi Valley that did not boast of their regular packets, often boats of great elaborateness of construction, giving rise on the Mississippi to the term "floating palaces". Too much praise cannot be given to this simple means of developing a great region; and, indeed, it has been asserted that the navigation on the Mississippi River had a direct influence on the result of the Civil War, as the control of its outlet by a separate government was viewed with great alarm by the residents of the upper valley who saw the possible adverse influence on their future development, and understood more clearly from this concrete argument their necessity for a continuance of the Union. The hazards of this sort of transportation, however, were very great, and the steam railroad was beginning to be a fierce competitor. Railroads found a fertile field in those areas where the rivers had already induced a new prosperity. As these railroads were developed throughout the country into that intricate network which is the pride of the nation to-day, little by little, the freight formerly handled by river was taken over by the railway lines, until now it is found that practically all the rivers of the great Mississippi Valley are dwindling in commerce in spite of the large expenditures for better channels that have been and are still being made. This falling off in usefulness is in the face of an enormous expansion in production of all kinds and a development of commerce that is unsurpassed in our history.

To illustrate, compare the commerce of the Kentucky River in 1892, amounting to 431 846 tons, with that of 1912 of 186 300 tons as shown by the Reports of the Chief of Engineers—a loss of nearly 60% in 20 years. In 1892, there were but five locks in the lower river, built by the State of Kentucky, permitting the use by vessels of depth from the Ohio to above Frankfort, 65 miles, whereas, in 1912 there were twelve locks furnishing a 6-ft. channel to a point 239.5 miles above the mouth. Thus it is found that, notwithstanding a great increase of facilities, there has been a decided reduction in commerce on this river. This condition was foretold by the engineer officer in charge of this work in 1895, but the progress of work could not be stopped, owing to the insistent demand of the locality for further improvement.

The Ohio River, which has held its commerce better than most of the interior streams on account of the coal deposits at its head-waters, has diminished in commerce from a total of 13 000 000 tons in 1905 to 8 618 000 tons in 1912, approximately 35% in 6 years. Between 80 and 90% of this business is in coal, which is carried on the river mostly at high stages, when works of improvement are not so necessary as at lower levels. The commerce of the Mississippi River at St. Louis has dwindled from 1 208 205 tons in 1892 to 265 720 tons in 1912, a loss of nearly 80% in 20 years. The commerce of the Green River in Kentucky has diminished from 462 208 tons in 1892 to 306 910 tons in 1912. On all the streams of the Mississippi Valley, there are few on which the commerce is not diminishing. The Great Kanawha barely holds its own in tonnage; the Cumberland River shows about 10% increase in traffic in 20 years; and on the Tennessee River there was also a considerable increase up to a few years ago, but a diminution since then. Almost all the others are losing their traffic. Such a record is indeed a disappointment to all those who were interested in fostering river commerce.

The Mississippi Valley is one of the richest and most productive areas in the world. The volume of commerce originating there is enormous, and has been rapidly increasing during the past 15 or 20 years, but, notwithstanding this growth, less and less is being carried by the rivers, and only those few streams which are not paralleled by railroads are still managing, with many a struggle, to maintain their former value to the public. Nowhere are the products of the farm more valuable; nowhere are the mines more productive; and nowhere

are the energy and capacity of its people excelled in all those various pursuits of wealth which are so numerous in this great area. If, therefore, the rivers anywhere might be expected to show increased usefulness as the country develops, it would certainly be in this region. It is, therefore, with keen disappointment that the opposite tendency is found to be growing more than ever apparent. Rivers in this valley were teeming with steamboats and barges within the memory of men still living. The arrival and departure of the large packets were events of much interest in the various towns where steamboat landings were scheduled, and boating was a well recognized industry, employing many men. To-day, however, the passenger traffic by river boat has almost disappeared, and bulky and slow freight forms the main part of what is left of a once flourishing business.

The explanation of this is not far to seek. The railway of to-day is well able to compete with the waterways at every turn. Combinations of many small and weak lines into through routes, and the extension of rail lines into every region where it seemed reasonable to expect a financial return, have developed what was a disconnected and feeble collection of roads into a systematized network of enormous value. The unfeeling and unrelenting competition which commenced first between river and rail lines and then continued among the various rail lines themselves, has forced an economy of operation and administration which has made the rail lines a giant in power and a miracle of usefulness. Within little more than a generation, the average cost of moving a ton of freight a mile in the United States has diminished from 7½ cents to about 7½ mills, a reduction of nine-tenths, and some of the coal roads having easy grades and flat curves boast that their tonnage cost has been cut to 2.3 mills per ton-mile. This is the harsh competition that rivers must meet. Railroads can be changed in location, and terminals can be placed wherever needed; cars can be switched from one line to another; spurs can lead to the point of destination without transhipment or breaking bulk, and distribution is simplified. On the other hand, waterways are fixed in location, require numerous expensive terminals, need constant improvement, and require steamboats and other craft of considerable cost. River lines have the advantage that their roadbed is provided by the Government. Railroads require large sums for general administration; for employees in yards and at stations; for maintenance of way and

equipment; for interest on their capital invested, and taxes. As the steamboat has no maintenance-of-way expense, it has a great advantage over the railroad in regard to fixed charges. Its interest on its cost has been estimated at 5%, insurance 8½%, and maintenance at 7½%, whereas the railroads must pay interest on the cost of the entire road, estimated at 5%, maintenance estimated at 2%, interest on the cost of equipment 5%, maintenance of equipment 10%, and insurance 3 per cent. Notwithstanding this unequal burden, railroads have out-distanced the river in economy of administration, and on most roads the fixed charges for interest on capital and maintenance are much less per ton-mile than those on a river packet. This indicates clearly the reason for the decline of the river commerce of St. Louis in 30 years from 2 120 825 tons in 1880 to 191 965 tons in 1910, a loss of about nine-tenths. During the 17-year period, from 1890 to 1906, the river commerce of that city dwindled from about 1 260 000 tons to about 317 000 tons, but in the same period the rail business increased from 15 000 000 to about 45 000 000 tons, or about 300%, according to the reports of the St. Louis Merchants Exchange. As a competitor, the Mississippi River has fallen from a position of preëminence to almost a negligible quantity.

Furthermore, of late years, a great decrease in the cost of handling freight on railroads has taken place, notwithstanding increases in cost of materials, labor, and taxes. On the other hand, an increase of at least 50% in steamboat operation costs has taken place, due to these same advances, but this has not been offset by reduced operating expenses. The river steamboat has not changed much in late years, and river terminals have nowhere been improved to a marked degree; certainly these improvements have not kept pace with the improvement of the channels by the Government. It has been stated that the stock in the largest company plying between Louisville and Cincinnati has fallen from a high premium to less than par within the last 25 years, a loss of more than 80 per cent.

Thus, it is seen that railways are driving steamboat commerce from most of our interior streams. This was started years ago by vicious hostility and unfair competition on the part of the railroads, but now the control by legal means of these unfair practices has left only general economic laws in operation, which, after all, form the safest basis for every industry of this kind.

The realization of the one-sidedness of this unequal struggle has of late years brought with it to river engineers a feeling that we are now far ahead of the demands of the present in many of our inland streams, and henceforth expenditures should be restricted to the barest necessities until the commercial development of the neighboring regions brings with it new demands. It seems not too strong a general statement to make that the navigational facilities on every stream on which commerce is now diminishing is far in advance of the present necessities, and that additional improvement at public expense should be withheld until such time as the economic pressure for additional transportation facilities becomes plainly manifest.

It seems reasonable to suppose that these tendencies toward diminution of river commerce are not necessarily permanent everywhere, for the commercial development of the areas contiguous to transportation lines is nearly always conspicuous, and frequently brings a need for additional facilities. New railroads are not being built so rapidly as a few years ago, and some day some of the interior streams may be expected to handle an increasing commerce in bulky freight where time of transit is not of great importance. This need for meeting the demands of expanding production may bring back some rivers to a new usefulness that cannot now be safely predicted.

Nor must we conclude that all our rivers are losing their value as commerce carriers, or that all are even diminishing in usefulness. Many streams outside the great central valley of the country, particularly those that empty in the harbors along the coast, and allow ocean-going vessels or coastwise ships to reach interior points, are showing very encouraging results. Some of those entering New York Harbor, for example, in the regions where population is dense, are very valuable and carry large quantities of commerce. In 1905 Arthur Kill had a commerce of 11 700 000 tons, valued at $265 000 000; and in 1911 it had a commerce of 30 500 000 tons, valued at $515 400 000.

In looking over the history of some of our rivers, it appears that they pass through several more or less indistinct stages of usefulness. First, while the country is comparatively undeveloped, and before the construction of railways has been begun, the rivers are found to be the best and cheapest lines of commerce. At such times the steamboat enterprise thrives and river commerce multiplies. In some cases

new land is opened to profitable cultivation, or new industries are encouraged. Later, towns spring up, and a whole region often receives a new impetus from its greater accessibility. Next comes the period when the results of this enterprise have brought about such a prosperous condition that the extension of railways is induced in the same territory. They, with their many advantages, then absorb most of the river transportation business. River commerce dwindles during this period, even though increase in production is noticeable on every hand. At such time discouragement may be felt by river advocates, because it seems that the rivers are not performing their wonted part in the upbuilding of the country through which they pass, for the part played by the streams is lost sight of in the general economic progress of the neighborhood.

This discouragement might be justified but for the third stage, which seems to be within the range of safe prediction for some of our internal waterways, especially if one judges by the rivers emptying into New York Harbor and those rivers like the Rhine flowing in congested communities. This third stage comes when the population and production have increased to such an extent that all lines of traffic are insured a large part in the transportation business of the locality. At such times the rivers again become useful and efficient. If we have proceeded too rapidly in the improvement of our interior rivers, there is still this hope of ultimate usefulness after the second stage has passed.

On the other hand, however, our harbors have had no such vicissitudes. Economy of ocean transportation has gradually forced shipowners to adopt deeper draft for their vessels. With the advent of steam, the depth of hull has steadily increased until now the largest ships are limited to few ports. The harbor having a depth of 25 ft., which was ample a few years ago, is now no longer sought by the large ocean vessels, unless the channels have been deepened to meet the new requirements. The entrance to New York Harbor, which had been ample for many years, has recently been dredged to a depth of 40 ft. to provide for this pronounced increase. Norfolk, Philadelphia, Savannah, Galveston, and many other harbors have also been compelled to provide for the increased draft of vessels.

The expansion of the railways throughout the interior, in many cases, has led directly to the greater development of the seaports.

Those who remember the almost feverish haste in building railways toward Galveston, when the harbor depths were first made ample for sea-going vessels, will see in that movement the importance of deep water when near productive areas.

The harbor, by its very nature, is a sort of terminal where products are exchanged between rail and water. Thus the railway, which is now having such an adverse effect on some of our interior rivers, has at the same time brought about a corresponding necessity for improvement in many of our harbors, and has contributed enormously to their value.

For example, in 1892, in New York Harbor, the exports and imports of foreign trade alone amounted to about 5 000 000 tons. In 1912, it totaled more than 14 000 000 tons, an increase of nearly 300 per cent. The total commerce by water was estimated in the 1906 census report at 114 000 000 tons. The harbor at Norfolk in 1892 had a commerce of 3 427 000 tons, whereas in 1912 it was more than 22 000 000 tons, an increase of nearly sixfold. Savannah in 1892 had about 2 000 000 tons of commerce, and in 1912 more than 3 120 000 tons, an increase of more than 50 per cent. Galveston in 1892 had a tonnage of 1 134 326 tons and in 1912 a total of 3 224 367 tons. These cases are probably the more conspicuous ones, but it may be accepted as a general rule that all the larger harbors have amply justified the expenditures made in providing better channels.

It is a wise policy, amply justified by experience, to have harbor facilities always a little in advance of the immediate necessities, and many of our seaports have responded admirably to efforts in this direction. It seems not unreasonable, however, to suggest that the liberal treatment which our interior streams have received could be restricted very materially, in the interest of economy.

On the whole, our expenditures for waterways have been of immeasurable benefit to the country at large. Never in our history has the volume of domestic and foreign commerce been so great, and never has the outlook for future increases been brighter. Our over-sea exports and imports are now growing enormously, and the prosperity of the land requires that there should be no restriction anywhere on account of inadequate channels. A practical test, almost infallible in its application, that will show whether a waterway project can be economically considered for further improvement, is a progressively

increasing commerce, and the measure of saving in cost of transportation will always be a guide as to the extent of work that is justified.

Thoughtful observers of our system of channel development have been impressed with the tendency of the Government to over-liberality in many instances, but the signs are multiplying that, in the near future, the public will demand a more rigid adherence to economic laws in the adoption of new projects.

DISCUSSION

F. LAVIS,* M. AM. SOC. C. E.—This paper calls very timely attention to one of the problems of the economics of transportation which is often misunderstood or entirely ignored. This misunderstanding or ignorance has resulted in the expenditure of a great deal of money without adequate benefit, and, not only has this been the case in the past, but it still continues. Mr. Lavis.

Lately, however, there has seemed to be a growing sense of the wastefulness of the so-called "pork barrel" methods of apportioning the expenditures for the improvements of rivers and harbors, though there is still far from a general appreciation of the necessity of the study of this whole question from the standpoint of true economy.

Our National policies in these matters seem to be too often developed in a generally haphazard manner, instead of being founded on sound economic laws; and, in the general development of the whole resources of their countries, certain European nations seem to be guided by more orderly processes of thought and action than we are; but, even in Germany, the greatest exponent of order and system, it is by no means a fact that the development of its waterways has resulted in unqualified success in the attainment of low costs of transportation thereon. It has frequently been assumed that this is the case, but it has been shown† that in Germany, as elsewhere, many of the items which should be included in the cost of water transportation have been omitted. Such success as has been attained, however, has been largely due to the fact that the matter of deciding on the projects on which public monies shall be spent has been generally left to a comparatively small body of men of training and experience in the particular class of work or problems involved, whereas, in the United States, this is generally decided by popular vote.

It is probably true that, in the development of their inland waterways, some of the European nations have attained a greater measure of success than we have, but in economic progress in the general realm of transportation, including that by railways, it is undeniable that we have far out-distanced the rest of the world. This success, generally speaking, has also been the result of policies formulated by a few men or by small groups who have brought our railroads to the high plane of efficiency on which they rest to-day. The word "rest" is used advisedly, as apparently the initiative which has achieved these results is now to be held in abeyance by the passing of the control of our transportation systems into the hands of those charged by the State with their regulation, and who have not the personal incentive to strive "to make a dollar earn the most interest."

* New York City.

† "Waterways and Railways," by H. G. Moulton.

Mr. Lavis.

Under our form of government it is inevitable that the final decision in regard to the expenditure of money for public improvements must rest on the approval of the people as a whole, and it is perhaps too much to expect that any large proportion of them can be so educated as to be able to form independent judgments on all matters of this kind, and especially is this so in the realm of economics, but, as a whole, they are generally quick to grasp the essentials if these are clearly presented to them.

The State of New York is just about completing a new canalized waterway from the Great Lakes to the navigable waters of the Hudson River at a cost of approximately \$130 000 000, or more. The construction of this canal was authorized by vote of the people of the State, only a few years ago, and apparently they were influenced by two arguments, both of which, in the speaker's opinion are fallacious. These arguments were:

First, that by providing competition the canal would compel the railroad lines to keep freight rates down.

Second, that the actual cost of transportation would be less by the canal than by railroad.

It does not seem necessary at this time to go into much detail to show how these arguments fail. The theory of competition is scarcely tenable, in view of the almost absolute control of railway rates by both State and Federal regulation. The actual cost of transportation by canal is only less when the interest on the investment in the canal and the cost of its administration and maintenance are ignored.

Taking into consideration the interest on the investment and the cost of maintenance, it has been shown* that the cost of transportation on the Erie Canal is about 8.6 mills per ton-mile, and this for low-grade, bulk freight alone, whereas, the receipts of the New York Central Railroad for all classes of freight were only 6.2 mills per ton-mile, indicating a considerably lower figure even than this for the low-grade, bulk freight.

It is also shown that the cost of construction of the old Erie Canal was not less than about \$165 000 per mile, and the new canal will cost double that, which may be compared with the capitalized value of the New York Central Railroad at about \$180 000 per mile.

Professor Engels, an eminent German authority, has stated† "that to improve a German river involved an expense equal to that of building a double-track railroad on easy gradients through a mountainous country", that is to say, from \$100 000 to \$150 000 per mile.

Even in regard to the Great Lakes of Canada, where water trans-

* "Cost of Transportation on the Erie Canal," *Bulletin*, Bureau of Railway Economics, 1911.

† *Engineering News*, January 21st, 1915, p. 116.

portation, except for the obstacles of the various locks, is somewhat on a par with ocean transportation, Sir Sandford Fleming, M. Am. Soc. C. E., a well-known authority on transportation, speaking of the then proposed Grand Trunk Pacific Railway, said:* Mr. Lavis.

"I have faith in an all-rail means of conveying the products of the farm to the seaboard * * *. Do not lose a day in proceeding with the location of a railway that will carry grain from the distant prairie fields to Quebec cheaper than by any other route whatever. * * * [The railway route] would be entirely free from the abnormal haste incident [to navigation on the Lakes] during the short period after harvest when it is possible to despatch the year's crop to market."

The question of the value of waterways for the purpose of providing competition for the railways is very ably discussed† by M. Colson, an eminent French authority, who uses as an illustration a specific case in France where the relations were apparently somewhat similar to those of the New York Central and the Erie Canal. The argument is paraphrased to fit this latter example, with which we are all familiar. It is argued that, if lower rates were the desired object, there is probably no doubt that if the New York Central had been presented with say one-half the cost of the Barge Canal, and half the estimated annual cost of administration and maintenance, it would have been glad to guarantee a freight rate of 2 mills per ton-mile (the actual net transportation cost on the canal) on such items of bulk freight as may be expected to pass through the canal and during the season the canal is open. The lower rates would thus be secured and the State would save many millions of dollars.

Probably, however, there would be a great outcry and protest against subsidizing a particular railroad to obtain a reduction of rates in a certain local section, without obtaining the same reduction in the rest of the country or State; yet this is really what is done, except that the cheap rates are obtained for a certain small part of the population by spending the money for a canal rather than by giving it to the railroad.

The importance of the study of the questions raised by the author is clearly apparent, especially in view of the existing financial and economic crisis. All who have broad knowledge of the newer civilizations, that is, those of North and South America, as well as Africa, Australia, etc., cannot but be impressed with the tremendously large number of important things to be done, and the difficulty of getting money enough to carry out even a small part of them, including often those of considerable merit. The Balkan War and the present European War, so far as their effects have any bearing on the financial situation, are but the culmination of a vast and far-reaching crisis

* *Engineering News*, November 5th, 1903, p. 416.

† *Bulletin*, International Railway Congress, November, 1913, p. 958.

Mr. Lavis. in the affairs of the world produced by the great demands for capital for legitimate and urgently required developments of new countries, and the totally inadequate supply. Therefore, it is doubly necessary now that the greatest care be exercised in selecting for development only those projects which are sound when judged from the broadest economic viewpoint.

The author has pointed out that, in technical skill, our engineers are quite the equals of any others. The speaker believes our Army engineers and many others are equally alive to the importance of the study of the commercial value of the projects presented for their consideration, but they are not always consulted in regard to this latter aspect, and often, if they are, their advice is ignored if it does not agree with political plans. It is also only too true that many engineers are so engrossed with the technical details of their work that they lose sight of its broader economic aspect, and it is well that our attention should be called, at this time, to this often very important phase of our work. In converting the forces of Nature to the use and benefit of mankind we must not lose sight of the fact that the extent and value of this use or benefit is most often measured in terms of dollars.

The present war has served to focus the attention of many of our people on our foreign trade, and, as usual, just as happens after an accident to a steamer or a train, the great mass of the people wakes up with a shock, the Public Service Commissions rush to make orders, the Legislatures and Congress pass or attempt to pass new laws, and now a great foreign commerce is to be created out of nothing in 24 hours. The speaker is in entire agreement with the conclusions of the author that it is time that we awoke to the wastefulness of much of the expenditure for improvements of rivers, and to the desirability of applying a larger proportion of the money toward increasing the facilities and depth of water in our ocean ports. He has not the faintest expectation that our foreign trade is going to jump into existence at once, and particularly not on account of the war, but it is going to grow because of economic causes, which had their origins long before there was a thought of war, at least in our minds.

We have heretofore been exporters principally of pastoral and agricultural products, and with these we have paid our debts abroad, that is, the interest on borrowed money, and for the luxuries and other goods we have bought for import. Our manufacturing, and therefore, necessarily food consuming, population, however, has been steadily growing, our surplus production of food available for export is getting less, and we must export manufactured articles to make up the deficiency. This economic condition has been growing visibly for the past two or three decades, and has been plainly foreseen by men of large vision, but the country and Congress have just awoke, and, as

usual, the true economic laws governing the situation are lost sight of in the excitement of trying to do something right away, instead of utilizing the brains of trained and experienced men to develop policies based on right principles and to carry them through. Mr. Lavis.

The development of our ports must be planned on an adequate basis to take care of a large overseas commerce, and our interior lines of communication must be developed to carry our products from one part of the country to another at the lowest possible cost per ton-mile. In the present state of the art, with possibly a very few exceptions, in respect to certain natural, deep, inland waterways, the railway is the best and cheapest means of inland transportation of which we have knowledge, and unless there is a distinct revolution along lines as yet unthought of, is likely to remain so for some time to come.

The author has referred to the three stages in the economic development of inland water routes, the first before the advent of the railroads, when they provided practically the only means of access and when, owing to the rapid growth of the country, they were prosperous; the second, the era when the railroads, in competition, almost entirely superseded water transport, and then thirdly, looking forward to a time when the waterways of the country may be utilized, to some such extent as they are in many parts of Europe, as almost equal in importance to the railways as lines of transportation for low-grade freight.

Of course, we cannot say that this third era will never develop in this country, but to the speaker it seems somewhat improbable that it will, if the development of the art of transportation by railway continues in the future to any such extent as it has in the past. In Europe the railways have never entirely superseded the rivers and canals, or offered the effective competition that they have in America, for it must be remembered that a great many of the mercantile and industrial centers of Europe were built up and established along the water routes long before the advent of railroads. Of course, many of the largest of the modern industrial and manufacturing enterprises have grown up coincidentally with the railroads, but the majority of them are only developments from those of an earlier period, built up alongside navigable rivers and canals.

Another point of considerable difference, is that in Europe, society and the industries, having been organized before the advent of the railroad as the primary agency of transport, were developed as a number of comparatively small self-sustaining groups, that is, self-sustaining so far as concerns the necessities of life. In the United States, however, the development of the railway as an efficient and cheap means of transport, reaching all parts of the country before the development of the principal industries, has enabled these latter to be located where the soil, climate, and other conditions were best suited

Mr. Lavis.

to them, and then to distribute the finished product to the point of consumption.

In the United States the country has been developed by the railways, and though many industrial establishments have been located so as to enjoy the benefits of both rail and water transportation, the railway is always of paramount importance. In spite of some adverse criticism of some details and of some of the figures used, the statement of James J. Hill, F. Am. Soc. C. E., that, in comparison with the rates of wages, the cost of freight transportation on the railways of this country is only from one-third to one-quarter as much as in Germany, is undoubtedly substantially correct. Taking into consideration, therefore, the limitations of flexibility of water transportation routes, the necessity that business shall come to them, and that they can seldom, if ever, be carried to the business, the fact that some cargoes, notably coal and grain, deteriorate from wet and dampness, that there are other inherent difficulties in connection with inland water transport, increased hazard, and insurance rates, and considering the efficiency of our railroads, there seems little reason to expect now that there will be any general development of fluvial or canalized inland transportation, unless there is a revolution in methods entirely beyond any present expectations, and which, therefore, it is useless to attempt to provide for.

The development on a large scale of our foreign commerce is inevitable if we are to progress as a nation in the future in any degree commensurate with our development in the past. To handle this foreign commerce, we must have adequate ports and harbors and an adequate and efficient system of transportation routes leading to these ports from all sections of the country. The railroads furnish the most efficient, that is, the best, cheapest, and quickest, means of transport we know of to-day, or which we can now imagine for the future; the logical developments, therefore, of the immediate present, and of the near future, are those calculated to increase the capacity and usefulness of our ports and of our railroads.

The foregoing is all more or less general, and the speaker would like to call attention to two specific problems, which have come under his observation and may serve to emphasize this necessity of developing a broad economic viewpoint in regard to the larger problems of transportation.

He was very much interested a few years ago in a study of the probable lines along which the development of the railway systems of the northern part of the Argentine would be carried out, and how this development would be affected by the value of the River Parana as a transportation route. The Parana is one of the great rivers of the world, and easily comparable with our own Mississippi, and like it heavily silt bearing.

Mr. Lavis.

An inspection of the map will show the general location. The Parana is navigable for ocean steamers (20 ft. draft) as far north as the City of Santa Fé. It has been proposed from time to time that it might be made navigable as far as the confluence of the Paraguay and the Alto Parana, at which point are the Cities of Corrientes and Resistencia. This point is now reached by flat-bottomed river steamers of the ordinary type, and at certain stages by keel ships of 10 to 12 ft. draft. The question which presents itself is in regard to the development of the country in the inverted triangle which has Santa Fé at its apex at the south, the Parana River as one long side on the east, the line from Santa Fé through Tucuman and Embarcacion as the western side, and the Pilcomayo River, on the northern boundary of the Argentine, as the base. This area is generally known as "The Chaco", the distance from Santa Fé to the northern end of it being approximately 1 000 km.

The Government of the Argentine has started the development of this area at its northern end by the construction of two lines of railway running a little north of west from two points on the Parana, Formosa and Resistencia, apparently on the theory that the products of the region will be brought to the river and so by the fluvial route to Buenos Aires, the capital. The products of this country will be principally agricultural and pastoral, requiring cheap transportation to move them. The speaker came to the conclusion that, if an efficient system of railway transportation were developed from the City of Santa Fé spreading out in a fan shape over the area as the country developed, and probably extended north of the Pilcomayo into Paraguay and Bolivia, in spite of the existence of the semi-navigable river, the products of this region could be brought to Santa Fé, and if necessary south of it, by this means, more cheaply than by any other, and that there would not then be any incentive to attempt the development of the river route for ocean-going vessels to points above Santa Fé. At present it takes from one to two weeks to reach the center of "The Chaco" from Buenos Aires, if dependence is placed on the fluvial routes to reach these Government railways. The same length of line built due north from, say, Tintina, would bring this section within less than 48 hours of the capital. Of course, there are many other factors affecting the situation, but the general problem is here outlined as a rather interesting example of those which engineers are sometimes called on to solve, which are rather outside of mere technical routine.

There is another large transportation project in which nearly all the countries of North and South America have been interested from time to time, the true solution of which, it seems to the speaker, is quite the opposite of those previously referred to, that is to say, that the railroad does not and, for a long time to come, would not afford

Mr. Lavis. as adequate or efficient transportation service as the water routes. The project is that of uniting all the countries of North, South, and Central America by the so-called Pan-American Railroad. Advocated first by the Pan-American Congress of 1892, which resulted in some extensive surveys, or rather reconnaissances, it has been fitfully kept alive, and we are told from time to time of the progress made toward its completion.

There is now communication from Canada, through the United States, to the southern border of Mexico. There connection is possible with the railway system of Guatemala, but this latter is of different gauge. At the southern end, both the Argentine and Chilean lines from their respective capitals, Buenos Aires and Santiago, except for a small gap in Bolivia near the Argentine border, have lines reaching to Bolivia and thence to Peru. The through routes, however, involve several changes by reason of breaks of gauge, and it is interesting to note that these international lines where they have been built along the route of the proposed Pan-American Railroad, and without other particular purpose, are all operated at a loss.

Groups of enthusiasts still gravely predict and advocate the completion of the line between Peru and Guatemala, for no imaginable reason except the purely sentimental one of being able to say that there is a continuous line of railway uniting all the countries from Canada and the United States to Chili and the Argentine. Apparently, the fact that there are almost as many gauges as countries does not seem to bother them at all. There is not anywhere along this route nearly enough business in sight, either freight or passenger, to warrant the construction of even a comparatively cheap road, and this proposed line, throughout nearly its whole length, would involve some of the heaviest and most difficult construction, in regions ranging from the densest tropical forests to altitudes of 12 000 to 14 000 ft. above sea level. That the railway lines of each country will eventually connect with each other, there is little reason to doubt, as each country develops its railway system and extends its lines to all its boundaries; but, as a through transportation route, it is entirely impractical. Generally speaking, both passengers and freight can be carried much more comfortably, expeditiously, and cheaply by ocean steamers, and if any money is available at any time in the proximate future in any of these countries, there are many other routes along which the construction of railways is far more urgently needed than that selected for the Pan-American lines. The continued advocacy of this scheme, mostly by people from the United States, seems therefore to be undesirable from the standpoint of true economic benefits to the countries concerned.

We are beginning to realize the necessity for the conservation of all our resources of every kind, and none other is so much in need of

conservation to-day as capital. As engineers, we can do our country and those dependent on us for advice and counsel no better service than to protest against wasteful expenditures for uneconomic projects, and use our whole influence to advise against any which, after careful consideration, seems to be economically unsound. Mr. Lavis.

Mr. Burgess.

H. Burgess,* M. Am. Soc. C. E. (by letter).—The author has given those reasons for the decrease in tonnage on interior waterways which appear to be the most important; but an additional cause may be mentioned, and that is the more favorable freight rate charged by railroads on any portion of their lines for which there is a competing water route. The result of the lower rates for towns connected by a water route is that "inland" towns must pay a portion of the rail transportation charge on freight received by river towns. These latter, therefore, receive the superior facilities offered by the rail route over carriage by water at practically what such freight would cost if carried by boat, and these superior facilities are obtained at the expense of the "inland" towns.

A railroad must collect a certain sum from its freight business for operating expenses and for return on investment, and there results an average charge per ton-mile; and if the ton-mile rate for portions of the line competing with the water routes is lower than this average, the rate must be above the average on other parts of the railroad system. The matter is complicated further, however, by the fact that this permitted lowering of the correct ton-mile charge actually succeeds in securing a greater total tonnage, with a lower average cost per ton-mile of freight carried; and, further, by the fact that water competition frequently means also either lower line grades or, for the same grade, a lower cost of construction per mile of road, with corresponding lower cost per mile for haulage or for the interest charge. These facts are probably responsible for the Interstate Commerce Commission permitting the railroad to collect from the inland shipper part of the freight charges which should be paid by the merchant of the river town. It would be quite difficult to arrive at a correct adjustment of the charges per ton-mile between the sections of the road which have water competition and those which have not, but there is little doubt that at present the inland town is paying for facilities furnished to the river town, and that the railroad is carrying freight destined under proper freight rates for the river route. It is not forgotten that many railroad men maintain that railroads can actually carry freight at a lower cost between terminals (not per mile, the rail route being much more direct as a rule) than is possible for the river route; but this position does not seem to be consistent with the fixing of lower than average freight rates where there is water competition. It is not probable that the railroad would propose its

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Mr. Burgess.

rates to be any lower than would be required to take away the business from the water route.

It is not easy to suggest an equitable remedy for this condition; but as long as the railroad is permitted to urge water competition as an excuse for rates below the average, the boat lines can expect to carry only a small portion of the tonnage which, under normal conditions, would fall to them, and the inland town can expect to be unfairly dealt with as to freight rates for benefits which are received by the river towns.

The author has mentioned the great improvement in facilities offered to shippers by the railroad in the past few decades, but has hardly brought out with sufficient emphasis the relatively slight increase in the facilities offered by the boat lines. Relatively, the water route has fallen far behind, in spite of the fact that both as to depth and width of channels the interior streams have been improved considerably since the date when the rail and the water route were about on a parity as to advantages to the shipper. This falling behind is due chiefly to the failure to push to quick completion the improvements of streams, a failure due, in turn, to the scattering of annual expenditures on many streams, instead of concentrating the available money on a few until the work on them is completed, and then following with the improvement of others. A stream which is improved in disconnected sections is little better as to facilities for navigation than in an unimproved state, and is very much like a railroad which is completed in detached sections. Or, possibly a better comparison would be with a railroad, the roadbed of which during from 4 to 6 months per year was interrupted at intervals with submersion by overflowing streams, providing continuous traffic facilities for from 6 to 8 months per year, and only for local traffic during the remaining months. A stream being canalized for barge traffic is not available for such traffic throughout the year until the entire improvement is completed, although, at times of high and mid-stream stages, the barges may be moved from end to end of the stream, while during the low-water season only local traffic is possible.

Taking a specific case, the Ohio River might be compared with the trunk line of a railroad, and its navigable tributaries to the branch lines serving to feed freight to the main line. Here, due to the scattering of appropriations for river improvements, the trunk line is usable for only a portion of the year, while some of the branch lines, as the Kanawha, the Kentucky, and the Green, are at all times available for traffic, and other branch lines, like the Cumberland and the Tennessee, have a year-round traffic only for disconnected sections. Because of these conditions, and of the declination of railroads to make joint rates over land-water routes and to interchange freight with boat lines, it is not surprising that commerce on these streams

Mr. Burgess.

is confined principally to the local movement of products, the origin and destination of which are on the river banks. What railroad system would be able to show high traffic development under conditions of complete interruption of traffic for long periods each year on its main line and most important branches, especially if other carriers would not agree to the interchange of freight on a reasonable basis? Under authority of the Panama Canal Act, the Interstate Commerce Commission can now make through rates for part rail and part water hauls; and interchange of traffic between rail and boat lines can be expected when the navigable condition of any stream is such as to warrant the exercise of the authority granted to the Commission.

When the improvement of an entire river system has been completed, with its trunk line and feeders reaching the important shipping points in its basin, and when the Interstate Commerce Commission has abolished the inequitable traffic rates heretofore permitted where there is water competition, and has directed through traffic arrangements for rail-water routing of freight, then there will undoubtedly be a considerable increase in the tonnage carried on the waterway, even without awaiting the "third stage" mentioned by the author; but whether, during his "second stage", the tonnage carried will be sufficient to justify the cost of the improvement of the streams, is another matter. The completion of the improvement of the river system and the regulation of freight rates are questions for the Law, and the beneficial results which will ensue is a question for the Prophets. In the absence of a true test, under conditions reasonably favorable to the interior waterways, there will remain great differences of opinion as to the utility of river improvement.

Mr. Pease.

H. T. Pease,* Assoc. M. Am. Soc. C. E. (by letter).—There is much to be said in favor of the author's argument for economy in rivers and harbors appropriations. For one thing, the political "pork barrel" feature should be eliminated. This is aside from the author's argument, but, nevertheless, has had to do with making the figures for such work as large as they are. However, omitting this feature, the argument still stands that much has been spent unwisely and prematurely for river and harbor improvement and, without doubt, much more might be accomplished with equal appropriations, if all the economic as well as purely engineering features were more judiciously analyzed before such appropriations were made.

There is no question that some waterways can never compete with railroads in the same territory. This is probably more nearly true of inland waterways than of those leading directly to the coast. However, the economic features of each case are the determining factors. The problem is to read them aright. Take, for instance, the Great Lakes: The tonnage on this great interior waterway is not decreasing,

* Deer Park, Wash.

Mr. Pease.

and the reasons are plain. The greatest wheat area of North America is at the head of the Lakes, and they lie in a direct line between that territory and the chief markets for wheat, the Eastern States and Europe. The wheat goes this way in spite of the fact that it has to be transhipped several times *en route*. Again, the greatest iron deposits of the world lie at one end of this waterway and great deposits of coal at the other. Therefore, the iron takes this route to the coal. These, however, are not the only determining features; the Lakes are broad and deep, and the draft and tonnage of Lake boats are almost as unlimited as those of sea-going vessels; moreover, the value of this route has been demonstrated, so that terminal facilities equal to those of any railroad have been provided.

The case of the Ohio River shows a contrast, both in economic features and in results. The Ohio runs the wrong way. If this river headed at Cairo and emptied into Delaware Bay, it would be to-day one of the principal routes of transportation of America. As it is, it heads in a manufacturing district and flows toward an agricultural district, and a comparatively small market for its manufactures at that. Agricultural products are heavier and more bulky than manufactures, and, in a scheme realizing the greatest economy, should move down stream, while the smaller cargoes of manufactured goods should move up stream.

The requisite for greatest economy in river transportation is that the agricultural and mining districts should be along the upper reaches of the river, and the manufacturing districts should lie along the lower portion, the port for foreign commerce being at the mouth. Gravity is an important factor where it can be utilized, for it is certainly cheap. Analogy to the modern factory may be drawn: The raw material goes in at the top, and gravity is utilized as far as possible.

The one big article of commerce which the Ohio could transport to advantage is coal, but the coal market is also in the other direction, to the north and east. It should be admitted, therefore, that at least a part of the great expenditure on the Ohio has not been warranted or is premature, and the same conclusion follows in the case of many other streams, notably the Mississippi.

An examination of the latest factors which will tend to change conditions may show a future for some of the streams at least, and give some assurance that all the work done on them has not been lost.

The Ohio and Mississippi are in the same category at present. There is no market farther down to be reached for the products at the head, and the route is not direct enough for these products to their present market, which is principally the Northeastern States and, in a smaller way, Europe. The great future market, however,

will be the lower Mississippi Valley itself. It is the greatest potential area for the production of raw products in North America, and the future will see the river lined with factories from St. Louis to the mouth. Then the Ohio will ship its coal down stream—there is little water power down there—the Northwest will send down its grain to feed the ever-increasing population, and the manufactured products will in part go back up the river and in part tranship for South America and Europe. Nor will it be simply a case of over-congestion of population forcing a part of the traffic to the river, but rather that the river itself will outdo all competitors in transporting such materials as are especially adapted to such methods of transportation. Mr. Pease.

It is not even too much to predict that the future will see a great, deep-channel waterway, aside from the river itself—which is impossible of control as anything but a shallow stream—in the river valley and extending far inland to some great central seaport of the United States.

J. H. BERNHARD,* ASSOC. M. AM. SOC. C. E. (by letter).—The author has given the typical view-point of the Army Engineer on river improvements, and, as Mr. Lavis has advanced the arguments generally given by the railroads, it may not be amiss to offer for consideration the observations of a water transportation engineer on this subject. Mr. Bernhard.

As the author claims that the river engineering of the United States is nowhere surpassed in its suitability for the work proposed, it may be justifiable to repeat a statement made by Charles S. Riché, M. Am. Soc. C. E., Lt.-Col., Corps of Engrs., U. S. A., who, in a little pamphlet published by him, and in an address delivered at the Inter-coastal Canal League meeting in Orange, Tex., in 1912, pointed out that there are forty-three different kinds of lock dimensions and twenty-six various channel dimensions in the United States. As stated by the writer, in an address before the River Terminal Conference in St. Louis, in April, 1914, "God alone could build a boat that would meet economically such a variance in conditions". The railroads, on the other hand, have a uniform track, and permit a uniform height, length, and width of car, so that a railroad car can go anywhere in the United States, Mexico, and Canada.

The author, remarking that the traffic on the majority of our inland water routes has decreased, states: "it is apparent that the present condition of our water routes, as to the depth and availability of their channels, is in advance of the use being made of them". Does this mean that, in his estimation, further improvement is not necessary? If this argument held true, the poorer the roadbed or the poorer the street, the less the reason for improvement. The author wishes us to believe such when he states, in his next sentence:

* New York City.

Mr. Bernhard.

"Whenever streams do not show an increasing traffic which is at least approaching the increase in commerce in the region traversed, it shows that the usefulness of the stream in question is declining, and the necessity for continuing large expenditures is open to doubt."

The writer's contention is that the only result of such decline should be the immediate investigation of its cause.

Although entirely in accord with Maj. Burgess, who places the cause of the decline on the railroad rate, the writer would like to add to this reason other equally important ones. Next to the rate-anarchism, poor river terminals, obsolete boats, the absence of proper connecting links between existing routes, poor navigation laws and regulations, Government assistance, banking facilities, and insurance rates are equally important as causes for the decline of water transportation.

The rate situation in the United States is a natural result of former railroad abuse. The rail rates form a most unnatural and impossible fabric based on two faulty foundations: first, "charge what the traffic will stand"; and second, "any rate to meet water competition".

It is almost entirely correct to say that there is no rail rate in the United States based on cost plus profit, although the aggregate—the gross receipts of the railroad—is calculated in that way. In the preliminary report just published by the Interstate Commerce Commission for the fiscal year ending June 30th, 1914, the average cost of operating, for the railroads in the United States, was given as 72.21% of the gross receipts, and the average receipts were 0.733 cent per ton-mile. The rail distance between New Orleans and St. Louis is 700 miles, which would give, as the average charge, based on 0.733 cent per ton-mile, $5.13. With these facts at hand, examine some of the following rates in effect between New Orleans and St. Louis:

The rate for bran is $2.10; beef, $5.00; beer, $4.20; bridge materials, $3.50; canned goods, $5.20; cement, $2.50; coffee, $4.60; condensed milk, $4.20; flour, $2.80; grain, $1.80; lumber, $4.00; molasses, $4.10; nails, $4.40; nitrate of soda, $3.40; paint, $4.60; rice, $4.80; salt, $4.30; soap, $4.60; sugar, $3.40; and vehicles, $5.00.

As will be seen from the foregoing, all these charges are well below the average, but then there might be water service between New Orleans and St. Louis.

Although rebates are forbidden by State or Federal statutes, railroads grant them quite as much to-day as at any time in the past, the only distinction being that formerly they were granted to individuals surreptitiously, and now they are granted to cities and towns openly; the former became illegal, the latter is legalized. These rebates arise from the unjust practice of railroads in basing rates to river points on water competition.

The railroads say they must be allowed to meet the competition of the boats, otherwise the boats will take their business between river points. The Interstate Commerce Commission admits this claim, and allows the railroads to haul freight to river points at a loss, and then recoup themselves for this loss by exorbitant charges to off-river points. From these facts, one of two things must be true: either the railroads lose money on all hauls to river points, or they get excessive returns on hauls to off-river points. It is plainly a rebate case—"robbing Peter to pay Paul". The river towns are having their freight hauled at the expense of the off-river towns, and pay for this privilege with the loss of still lower rates by boats; and the public loses all around. Mr. Bernhard.

Examples of actually existing rates will be illuminating: first-class freight is shipped from New Orleans to Memphis, 396 miles, for $9.00 per ton, or 22.8 mills per ton-mile. To ship the same commodities from New Orleans to Amite, La., 68 miles, the cost is $10.40 per ton, or 153 mills per ton-mile. The citizens of Amite pay more than six and one-half times as much per ton-mile for their freight charges as the more fortunately situated residents of Memphis. Suppose the Government should charge the Amite people 13 cents for postage on a letter, but those in Memphis only 2 cents!

To ship a barrel of flour from New Orleans to Amite costs 34 cents, or 5 mills per barrel-mile; to ship one from New Orleans to Memphis costs 25 cents, or 0.63 mill per barrel-mile. For carrying a barrel of flour 1 mile, the cost to the consumer in Amite is eight times as much as to the consumer in Memphis. First-class freight from New Orleans to Memphis (396 miles), is 45 cents per 100 lb.; from New Orleans to Opelika, Ala. (386 miles), it is 99 cents per 100 lb., more than twice as much, for a shorter distance, to the off-river point. These are rates from one place to another which can be reached by water, compared with rates from the same place to another which can only be reached by rail.

Table 1 will illustrate the "rate-wall" that hinders the freight movement to the river.

Although the distance from Birmingham to Tuscaloosa is about one-eighth of that from Birmingham to New Orleans, the average freight rate is about one-half of that to New Orleans; but Tuscaloosa is on a navigable water, by which New Orleans, Mobile, or other cities may be reached.

As a third example, the variation in rates, over the same distance, and over the same road, will be shown. This is simply one example where hundreds could be given. Flour for domestic use moves from St. Paul to New Orleans for $5.50 per ton; from New Orleans to St. Paul (same routing), for $9.80 per ton; but one is down stream and the higher rates are up stream. "No", says the railroad, "it is because the natural movement of flour is from St. Paul to New

Mr. Bernhard. Orleans". The latter is true, because the rates forbid any other movement; but great would be the tonnage of finer winter flour that would move up stream if the rates were not prohibitive. In any case, why should there be any difference in rates over the same distance and the same railroad? Did not Congress long ago rule that rates under the same conditions should be equal, regardless of the tonnage shipped by the individual shipper? Why is this not equally true for one community against another? Had this flour been moved by water, and had it been meant for export, it would have been more cheaply loaded from barge to ship than to warehouses in the cities, therefore, the railroads have a third rate, which is the "export rate". On flour from St. Paul to New Orleans the rate is $3.90, or 2.6 mills, per ton-mile.

TABLE 1.—Comparison of Freight Rates from Birmingham to New Orleans and to Tuscaloosa.

Commodity.	Distance from Birmingham:		Rate from Birmingham:		Ratio. %
	To New Orleans.	To Tuscaloosa.	To New Orleans.	To Tuscaloosa.	
	418 miles	56 miles.	....	....	13½
Sugar	....	....	$3.40	$4.80	140
Coffee	....	....	7.00	3.00	43
Rice	....	....	5.00	3.90	78
Molasses	....	....	4.40	1.80	41
Dried fruit	....	....	11.00	1.60	17½
Peas and beans	....	....	11.00	1.50	13½
Hay	....	....	3.60	1.40	39
Grain	....	....	3.60	1.40	39
Flour	....	....	4.40	1.60	36½
Coal	....	....	1.25	0.80	64
Coke	....	....	1.75	0.80	46
Wire and nails	....	....	3.20	2.00	62½
Pig iron	....	....	3.00	1.00	33⅓
Steel rails	....	....	3.00	1.50	50
Cast-iron pipe	....	....	3.00	1.50	50
Machinery	....	....	5.40	3.90	72

The foregoing cases could be multiplied indefinitely, as they are representative of thousands of others, all of which show the injustice suffered by land communities in the matter of freight rates.

Though the paralleling railroads of our inland waterways are strained to their utmost to carry the freight offered at each crop-moving season, the waterways themselves are vast expanses of idleness.

A mistake is made by the public in assuming that it is always the river channel that causes this idleness. Nothing could be farther from the truth. To-day the Mississippi, from St. Louis to its mouth, affords a channel which is the best to be found on any stream in the world, unless one takes the Amazon or the Congo into account; and see its emptiness! An 8-ft. channel is all that the most efficient service requires. The Government works unremittingly to develop

waterways, only to see the water-borne traffic on inland rivers grow less as the years go by, not due chiefly to the inadequate depth of the channels, but to this rate-making anarchism; and until the idea that the principal function of inland water channels is to regulate the rates for rail transportation has been untaught, or made unnecessary by just rates, we will not see great river traffic. Mr. Bernhard.

Still the average "river-man" will insist that the poor condition of the channels keeps our inland waters idle. This is preposterous; the Rhine could never compare with the Mississippi in its advantages for transportation; its channel is narrower and shallower, more changeable, the current is swifter, and ice is known in the winter over its entire navigable length to its very mouth, yet, in 1913, more than 97 000 vessels passed the Dutch and German frontier on the Rhine, which means a vessel every 5 min. for the entire year. In order to illustrate this point clearly, page 109 of the English translation of the Trade Report of the Rotterdam Chamber of Commerce for 1913 will be quoted:

"We could almost suffice by referring to our review covering 1912, seeing the slight difference between 1913 and its predecessor.

"Arrivals are plentiful till the autumn, and the water level was satisfactory during the whole year. During only 9 days, from the 30th of October to the 7th of November, a short period of low water on the Rhine was witnessed when the water mark recorded a level of 1.20 m. (4 ft.), and below that to 1.14 m. (3 ft. 9 in.).

"The not normalized lower part of the river was, as a channel, in a rather favorable condition, the occurring sand deposits were for the greater part removed by dredging operations, and by altering the fairway in the second half of the year a sufficient channel was maintained.

"The normalized upper part was satisfactory as a fairway, although on several occasions dredging work had to be started near the entrance in order to maintain a sufficient channel from the regulated Maas to the Upper Merwede.

"The average Rhine freights were, during the year 1913, for cargoes of 800 to 1 000 tons, including tonnage; or ores and other crude commodities to Ruhrort, Disburg, Hockfeld, Alsum, Walsum, or Rheinhausen, $0.26 per ton, or 1.2 mills per ton-mile. For ores (copper ore, phosphate, etc.) to the Upper Rhine and Mainz stations, $0.56 per ton or 1.1 mills per ton-mile.

"On the whole, the freights were rather satisfactory, as was also the water level, which only from the middle of October to the beginning of November was low. The total tonnage across the German-Netherland frontiers was:

	1912.	1913.	More.	Less.
With Netherlands....	25 619 771.0	27 941 378.0	2 321 587.0	....
With Belgium.......	8 523 472.0	9 073 140.5	549 668.5	....
Rhine sea traffic.....	485 117.5	514 634.5	29 517.0	....
	34 628 360.5	37 529 153.0	2 900 729.5	....

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Tables 2 and 3 give a comparison of the Rhine traffic during the last 10 years.

TABLE 2.—RHINE TRAFFIC.

Year.	Rotterdam.	Amsterdam.	Belgium.
1904	10 684 261 tons	428 589 tons	4 104 306 tons
1905	12 771 307 "	478 320 "	4 435 580 "
1906	13 357 575 "	538 945 "	4 821 229 "
1907	14 762 526 "	597 518 "	4 987 736 "
1908	12 938 898 "	716 035 "	5 018 609 "
1909	15 134 175 "	798 535 "	6 205 324 "
1910	17 663 521 "	990 927 "	7 727 219 "
1911	19 042 847 "	1 042 603 "	7 956 855 "
1912	20 818 991 "	1 360 280 "	8 523 472 "
1913	22 764 241 "	1 531 772 "	9 073 140 "

TABLE 3.—NUMBER OF VESSELS PASSING LOBITH, TO AND FROM GERMANY.

Year.	Vessels.	Number of these vessels under the Dutch flag.
1904	67 519	46 584
1905	72 029	48 941
1906	75 306	49 821
1907	79 640	52 508
1908	71 206	47 021
1909	77 909	50 692
1910	85 372	55 740
1911	90 129	59 122
1912	91 904	58 978
1913	96 768	62 249

Compare the foregoing rates with the statement of Col. Harts to the effect that the average cost of moving a ton of freight a mile in the United States is 7.3 mills, and that some coal roads, having easy grades and flat curves, boast that their tonnage cost has been cut to 2.3 mills per ton-mile. Now, considering that the distance between Minneapolis and New Orleans by rail is 1 300 miles, this would mean that the average charge should be $9.50 per ton, and even when calculated at the lowest cost when moving coal, on the best roads, at 2.3 mills per ton-mile, it would be $3 per ton, though flour—which, as far as moving is concerned, is quite different from coal, being very susceptible to moisture and odors, and only handled in box cars—has a rate of $3.90 per ton from Minneapolis to New Orleans. That this rate is below cost will be clear. Somebody else pays for this loss; of course—the interior territories; but is it then just to say that the rivers have lost their usefulness? Would it not be more correct to state that, by reason of artificial barriers, transportation by water in the United States cannot compete economically with the railroads? These artificial obstructions, however, devised by railroads and perpetuated by the Interstate Commerce Commission, do not mean economy to the

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country, but the country at large, by excessive high inland rates, pays the penalty for the idleness of the water routes.

Even railroad officials are ready now to admit that their rail rates are less than what they should be along the water routes; they also admit that their rates on bulk commodities are profitless.

On April 3d, Mr. W. M. Rhett, Assistant Freight Traffic Manager of the Illinois Central Railroad, testified at the Interstate Commerce Commission hearing on the "Western Rate Case", that his road was handling many commodities, such as potash, clay, glycerine, pig lead, linoleum, vegetable tallow, rice, sisal grass, earthenware, hemp, and manila at a profitless rate. He mentioned only a few commodities; the writer could have named many more, and far more important ones. Undoubtedly, Mr. Rhett knows them, but does not feel inclined to have the rate of these particular commodities increased because they would move quickly by water. He was asking for an increase ranging from 10 cents to $1.00 per ton. Only these proposed increases, according to him, would enable the roads to continue business in these commodities. Even to bulk commodity rates, objections are raised. On the same day (April 3d) Mr. B. J. Rowe, Coal Traffic Manager of the Illinois Central Railroad, testified before the Interstate Commerce Commission, at the same hearing, that they would have to increase their rates at an average of 7.95 cents per ton in the western territories. All this came only shortly after the many "convincing" arguments of the railroads as to why their rates should be reduced, which "convincing" evidence moved the Interstate Commerce Commission to grant their request. (Intermountain Case.) The railroads were permitted to decrease their rates to meet competition created by the Panama Canal, with the result that the rates from coast to coast were lowered; but the transcontinental schedule for interior points was maintained. It is a ridiculous argument, if carefully analyzed. Suppose a Standard Oil trust came before the United States and asked permission to reduce its prices continually, even below the cost of production, merely to meet the competition of an independent firm. Why, it would be the clearest infringement of the Anti-Trust Law; yet the railroads, by permission of the Interstate Commerce Commission, are permitted to reduce their rates in order to offset canal competition, to create which the country has expended $400 000 000.

Follow this argument to its rational conclusion. Suppose the railroads lowered their rates—which is clearly the aim—to such a point as would enable them to compete successfully with water transportation through the Panama Canal; then the freight would continue to move by rail from coast to coast, and the Panama Canal, so far as interior freight movement is concerned, would be idle. The railroads would either have made impossible profits before the opening of the Panama Canal (and their financial reports belie this), or they are now

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suffering heavy financial losses, on account of the lowering of these coast to coast rates. Therefore, the natural and logical consequence of the ruling of the Interstate Commerce Commission is that the $400 000 000 for the Panama Canal is spent in vain, its only result being a heavy loss to our trunk lines. The writer goes a step farther, and claims that, in consequence of the ruling of the Interstate Commerce Commission, a law should be made either forbidding the use of the Panama Canal to coastwise traffic or creating prohibitive tolls, thus permitting the railroads to do business on the old basis, as the United States cannot afford to have railroad capital unprofitably invested.

It is the opinion of the Commission that, had the railroads been denied their request for permission to re-adjust rates to meet part way the lower charges *via* the Canal, there was "grave reason to think that the Atlantic Seaboard in the future would have supplied by water the Pacific Coast with the commodities in question, and that many industries in the neighborhood of Chicago would have either lost their Pacific Coast customers or have been compelled to migrate nearer to the Atlantic Seaboard" (or, the writer would add, be forced to use the Mississippi River).

Under the original order of the Intermountain Case, the carriers from the Missouri River westward were forbidden to charge more to an intermountain point than to a Pacific terminal. East of the Missouri River, the stringency of the rule was abated, so that from Chicago to intermountain points the excess charge permitted over the rate to the Pacific terminals was 7%; from Pittsburgh, 15%, and from the Atlantic Seaboard, 25 per cent. The shrinkage of rates from New York to San Francisco by way of the Canal put the transcontinental carriers in serious straits. On certain heavy commodities, moving largely by water, a serious loss in through earnings was inevitable, if the roads reduced their rates to the Pacific Coast to compete with the lower water rates. In addition to this loss on through revenues, the Commission points out, the carriers would have had to take a double loss in their revenues to intermountain points: first, because the intermountain rates would have to be lowered; and second, because the percentages over the terminal rates would have been calculated on a lower basis.

Additional relief was sought by the carriers as to the rates on about 100 carload-commodity items and about 50 less than carload items. These carload rates to the Pacific Coast range from 55 to 95 cents per 100 lb., and the less than carload-commodity rates range from $1.10 to $1.75 per 100 lb. These commodities are such as originate in large quantities at or near the Atlantic Seaboard, and are particularly adapted to water transportation, on which the rates are relatively low.

On about 25 items which move in carloads from the Missouri River to the Pacific Coast at rates of less than 75 cents per 100 lb., carriers are permitted to establish the rates proposed to the California terminals, and to continue rates to intermediate points not higher than 75 cents per 100 lb. On all other traffic, rates from the Missouri River to the Pacific Coast must be carried as maxima at intermediate points. Mr. Bernhard.

From Chicago, Buffalo, and New York, carriers are permitted to carry carload rates to intermediate points at 15, 25, and 35 cents, respectively, higher than from the Missouri River to the same destinations.

Less than carload-commodity rates on articles classed as first or second-class in western classification, which are less than $1.50 per 100 lb. from the Missouri River to the Pacific Coast, may be exceeded at intermediate points, but the rates on such articles to such intermediate points must not exceed $1.50 per 100 lb.

Less than carload-commodity rates on articles classified as third-class or lower in western classification, which are less than $1.25 per 100 lb. from the Missouri River to the Pacific Coast, may be exceeded at intermediate points, but the rates on such articles to such intermediate points must not exceed $1.25 per 100 lb.

Less than carload-commodity rates, from Chicago, Pittsburgh, and New York, to intermediate points, may exceed the rates from the Missouri River to the same destination by 25, 40, and 55 cents, respectively.

Carload rates on coal and pig iron may be less to the Pacific Coast than to intermediate points, but the rates on such articles to the higher-rated intermediate points must not exceed 5 mills per ton-mile.

A suggestion is made that rates to intermediate points near Pacific Coast terminals may be made by adding to the rates to the terminals something less than the locals from terminals to destination, or by making basing rates to the terminals less than the local rates to such points, to be used in connection with the local rates from the terminals to destination in making through rates from eastern points to points intermediate to the terminals.

The Pacific Coast terminals to which these rates will apply are the points at which the Atlantic-Pacific steamships deliver their freight.

In December, 1914, Commissioner Harlan, in his dissenting opinion on the railroad rate increase, said:

"I cannot but think that a general increase in the standard rates of this country, while the rate structures of these carriers remain full of inconsistencies, discriminations, and wrongful practices that deplete their revenues, is morally wrong; that the placing of additional burden on the Interstate Commerce that is not also placed upon the State, is also wrong, and that the course approved in the supplemental report will ultimately be as disastrous to the carriers themselves as it will be harmful to the general interest."

Mr. Bernhard.

Commissioner Clements, in his dissenting opinion on the same question, said:

"I cannot but regard the action now taken by the Commission as out of harmony with the spirit and purpose of the law, and as taking a step that leads away from the sound principles necessary to conserve the ends of justice. If the legislative authority of the Commission is as broad and unrestricted as this, then I must confess that I have gravely misunderstood the limitations upon our statutory authority, as well as the constitutional power of Congress to delegate its legislative power. If now, to strengthen and maintain the credit of the carriers, regardless of the causes of its exhaustion or impairment, and with the application of the usual tests of reasonableness, these increases are justified, then it seems to me that we are only at the beginning of what I fear will be a train of demoralized results, disappointing and embarrassing to all concerned. It is by no means certain that it would not in the long run be cheaper to the public to guarantee the bonds of the weak roads unable to meet their obligations, rather than for all to try to take care of them by increasing their rates, which inure to the strong roads as well as to the weaker."

The New York Times,* gave the railroad point of view, in the following editorial, which, when read by engineers trained as such, will appear to them to be surcharged with inconsistencies and improbabilities:

"The Interstate Commerce Commission has allowed the railways doing transcontinental business to modify the established rates in a manner to meet the competition by the canal route. The spokesman for the railways most concerned is pleased, but only moderately so. The railways would have liked a free hand to meet competition without regard to other rates, whereas the Commission still insists upon preserving a relation between the through rates and the intermediate rates. Moreover, only some twenty-five commodities are concerned in the decision. Its effect regarding earnings, therefore, is not so great as might be imagined, but the decision is important for another reason.

"In principle it is the adoption of the railway theory of rate-making over the ideal or Commission theory upon some of the most contentious points in the long controversy. After a quarter of a century of regulation, the Commission finds itself under the compulsion of submitting to a control stronger than the law's. The economic consequences of its methods were so intolerable that it has been found necessary to defer to the methods which it was resolved to abolish. Having begun, it is difficult to say where the process of reconsideration will stop. The first step was the most difficult to take, and the second step is longer than the first, with others just ahead.

"Congress decreed that the railways should not charge more for a short haul than for a long haul which included the short haul. A higher charge for a shorter distance is indeed illogical, if there is nothing more to be considered. Congress, to provide for other con-

* February 13th, 1915, after the decision of the Interstate Commerce Commission became known.

ditions, gave the Commission discretion to grant exceptions when conditions were unlike. The Commission made a decision which it now modifies. The length of the haul is subordinated to the necessities of competition by water, and the charge to intermediate points is to be calculated by adding to the through or competitive rate a charge for the 'back-haul', that is, the haul back from the coast to the point passed on the way to the coast. The railways are not allowed to have an entirely free hand. The Commission itself names rates not to be exceeded, as a precaution against the railways raising the intermediate rates unreasonably in order to compensate for the reduction enforced by water competition. The main thing is that the earlier decision under the perplexing long and short haul clause is set aside, and that the effect of competition is recognized. It will be easier to break the rule on another occasion, and the tyranny of a theoretical relation of rates established by the Commission itself is shaken." Mr. Bernhard.

The second reason given by the writer for the decline of water transportation in the United States is the obsoleteness of the river craft. In order that it may be clear how obsolete the present river boat is, there are submitted some comparative data between a modern self-propelled steel barge which operated on the Mississippi River between New Orleans and St. Paul, and the prevailing type of Mississippi River boat, the like of which, in one instance, constitutes the entire fleet of one of the least inefficient companies on the Mississippi River. The 1 000-ton, self-propelled, twin-screw, shallow-draft, steel barge, operated by two gas-producer engines, has been described in a recent technical paper.* The barge had a crew of 7 men and a speed of 8 miles per hour when loaded with only 1 000 tons of cargo. It consumed fuel at the rate of 30 cents per hour, proceeding quietly, with the total absence of wave, so that a rowboat would hardly rock in its wake.

The cost of this 1 000-ton barge complete, with all equipment, was \$32 000. The average cost of a Mississippi River boat of like capacity is from \$60 000 to \$70 000. Such Mississippi River craft has an average crew of 54. Insurance on the barge was 4% for fire and marine; on the steamboat it is 12 to 15%; fuel consumption on the barge was 30 cents per hour, on the Mississippi River craft it is \$4 per hour, both boats traveling at a speed of 8 miles per hour.

The total cost of operation of the barge, including 6% interest and all other charges (wages, fuel, supplies, subsistence, repairs, insurance, depreciation, etc.), was \$2 100 per month; on the Mississippi River steamboat it is \$8 100 per month.

That these barges can be constructed strongly might be proved by the fact that the Lloyds Company classified one of this type recently as "A-1" for ocean travel. Compare with this the remarkable

* *Engineering News*, December 4th, 1913.

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weakness of the present steamboat, which might easily be described as a raft consisting of several thousand dollars worth of junk; in fact, the prevailing Mississippi River steamboats are built of wood, tin, shingles, canvas, and twine.

In addition to their weakness, these boats are distinguished by the frivolity of their management. In the past, races have been of frequent occurrence, and descriptions would be comical were they not pathetic sketches of the lack of efficiency caused by trifling with serious navigation.

"No", the river man objects, "our boats are not obsolete; we have kept pace with the times, and our boats are to-day a great improvement on those of the Fifties." The other day a captain said to the writer:

"You are not familiar with the improvements we have brought about on our river craft; our steamers are now almost fire-proof; we have patent tapered floors, balanced rudders, steel and asphalt decks; we have the improved tandem-compound or cross-compound piston-valve engine, steam capstan and steering machine, a locomotive type of boiler, wire railings and tiller ropes, electric lights, combined breeching shield and furnace damper, insulated pipe coverings, and many more important improvements that substantiate the statement that our present side and stern-wheel steamers are the best type of boats for river craft in the world."

This enthusiastic description of the important improvements of the present Mississippi River boat, from one of its ardent champions, illustrates, better than anything the writer can say, their total obsolescence and inefficiency. However, this is not the only difficulty under which inland navigation in the United States is laboring at the present time. Added difficulties are the navigation laws and regulations. With a traffic of 50 000 000 tons on the Rhine, and a proportionately heavy traffic on the other inland waters of Holland, Germany, Belgium, and France, it has never been found necessary to make it compulsory to employ Government licensed labor; yet, with the small tonnage transported on the waters of the United States, the Government enforces the employment of licensed captains, pilots, and engineers, and now even of able-bodied sailors; and the latter licensed persons are permitted to form unions, and thereby create what might be the most absolute monopoly one can dream of. To this must be added the fact that, to all intents and purposes, there is a surplus of demand over supply of available labor in the market, so that one might frequently have to pay heavy traveling expenses to get a desirable crew together.

Another trouble on our alluvial streams is the so-called "Aids to Navigation". Great difficulties are caused by the tardy adjustment of the Government channel lights, which adjustment, as a rule, is

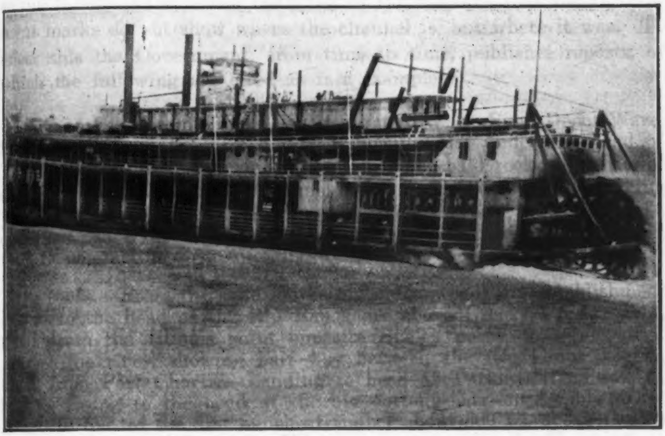

FIG. 1.—TYPICAL RIVER STEAMER. CUMBERSOME, WEAK, AND WASTEFUL.

FIG. 2.—SELF-PROPELLED BARGE OF SAME SIZE AS STEAMER SHOWN BY FIG. 1, BUT WITH THREE TIMES AS GREAT CARRYING CAPACITY AT HALF THE COST, AND WITH LESS NOISE, SMOKE, AND WASH.

so much behind the actual changing of the channel, that the Government marks do not show where the channel is, but where it was. To offset this the Government, from time to time, publishes reports, of which the following are cited as fair examples: Mr. Bernhard.

"*Magnolia.*—From Crooks Lower Light to White Sand Depot Warehouse, until from the first big tree showing just under the head of Turkey Island, to the upper side of the upper bald rock on the lower side of Magnolia Hollow. (Channel depth 12 ft. both.) This is taking you out square under an upper reef. Over this way, until from the corner of the Willows opposite Crooks Lower Light to Magnolia Light (16½ ft. channel depth).

"*Crooks.*—From the head of the big cottonwoods on Establishment Island to the head of the Rip Rap shore above Crooks Upper Light, until from the Illinois point opposite Snell's point, to straw stack nearest the river, showing half way between the two Crook Lights, until from Fort Chartres Landing to head of Turkey Island (12 ft. both). Going to the head of the big cottonwoods on Establishment Island to foot of the first bunch of timber about 300 yd. above Crooks Upper Light, until from the Illinois point opposite Snell's point, to strawstack nearest the river showing half way between the two Crook Lights, until from Fort Chartres Landing to head of Turkey Island (12 ft. both).

"*Montesano.*—From the field on the hill above Fines Fluff Light to Jim Smith's Upper Light (12 ft.) until from the middle of the middle bar abreast of Fines Fluff to Water's Point Light. Down this way, passing the rock pile about 150 yd. off, until from what shows the head of timber on the Illinois side above Jim Smith's rock light, to the first white house below foot of Montesano Light (no less than 10½ ft. in the above marks)."

In discussing this matter with George R. Putnam, M. Am. Soc. C. E., Commissioner of Lighthouses, the writer did not receive any other argument against the installation of the proper system than financial considerations.

Now, considering the fact that approximately $100 000 is spent yearly on the channel lights of the Lower Mississippi River alone, mostly showing where the channel was and not where it is, the writer asks: Is a slight increase, say of 15 or 20% of the above cost, objectionable, if it will secure a light system which shows always where the channel is, so that navigators may navigate on these lights, instead of avoiding them? Is it just to criticize the navigators for a decline in their business under such conditions? What railroad could exist to-day if, instead of sending track walkers daily over the roadbed, it employed a number of high salaried officials to go over the entire track once or twice a year, with a special train, and on returning home issue a report, after which another train would be sent out with the necessary crew

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and high officials to make repairs, as suggested by this report? Yet this is the very method adopted for channel lights on the Mississippi. Is the author correct in stating that the dwindling in traffic is proof that the availability of the channels is in advance of the use being made of them?

Another obstruction to economical navigation is caused by the total absence of banking accommodations. Bills of lading are frequently considered as bank securities unless they happen to be for freight on inland water craft, and then the bank refuses them. With a blind mule or an automobile as security, one can get at least a certain percentage of its value as a loan, but with the cheapest tool of transportation—a river boat—one might well have to sift the entire United States in order to find a single man or bank willing to furnish reasonable banking accommodations.

Yet the combined ship mortgage banks in Holland, including all their losses, have made an average profit of 18% per year, during the last 5 years. In Holland, with a boat as collateral, one may obtain, almost invariably, up to two-thirds of its estimated value, as a loan, at a rate which would show a total expense of 6½ per cent.

Another important obstacle is the insurance rates. A barge carried from New Orleans to St. Paul, 1 921 miles, 90 tons of coffee, on which a freight rate of $4.25 per ton was paid. The insurance rate was 90 cents per $100 value, or $2.70 per ton, and the terminal charges were 43 cents, leaving a net total of $1.12 for carrying the ton 1 921 miles.

This rate of insurance is actually several times larger than insurance on the same commodity from Brazil to St. Paul, provided it is moved by rail from New Orleans to St. Paul, and why is this?

There are three reasons for this flagrant injustice: *A.*—The small inland water-borne traffic makes it no fair average risk; the laws of average are hardly applicable for this limited movement. *B.*—The insurance companies, like most of the people in the United States, do not take water transportation seriously; they are not willing to accept the movement as a growing one, as one with a bright future; and, therefore, they do not care to foster it because they only see possible losses, and not even the promise of a future. *C.*—Most of the insurance companies have invested their securities in railroad bonds, which circumstance seems to strengthen their belief that water transportation should not be taken seriously; at least, they have no burning desire to foster it. Some may feel inclined to add a fourth reason by speaking of the "River Pool", but the writer prefers to ignore this factor, as he has been successful, in his business, in doing without it, and therefore knows little about the working of this combine.

The greatest obstacle and handicap to inland navigation in the United States to-day, however, is the absence of terminals. This is especially true of the Mississippi River, which is cited because its

conditions are typical of the entire United States. On the Mississippi we are still living in the ante-bellum days, where the drowsy negro roustabouts draw the preposterous wages of from $80 to $100 per month, barring modern loading and unloading apparatus. We on the Mississippi are still content with mud levees, slippery and cumbersome, and feel fully compensated for economical inefficiency by the great smokestacks of our steamers belching forth clouds of smoke, in their large whistles which can be heard for miles, and in the wheels that beat the water up in waves higher than a man—steamboats having daily expenses which surpass the weekly expenses of more modern craft of larger capacity. So important is the terminal question that it may be said that the problem of inland transportation is more a shore than a water problem. Almost invariably the shore expenses are higher than the actual cost of transportation from harbor to harbor. Mr. Bernhard.

With the present river terminals, the height of inefficiency seems to have been reached. The shipper must carry his freight by wagon over poor roads, move it somehow down a poorly paved and usually very steep bank, laboriously unload it by hand, roll it to the edge of a muddy levee, watch it until the steamboat may arrive, when it is carried piece by piece and stowed away under the towering superstructure of an American steamboat, amid a forest of deck supports; and the process must be reversed at the other end of the voyage. The only exception to these conditions is the City of New Orleans, where covered sheds are provided and where mechanical loading and unloading devices are now tried. Economical navigation of the stream, or efficiency of the carrier, are greatly handicapped under such conditions.

The elimination of the steamboat gangplank, as the highway of water traffic, and the abolition of the roustabouts are necessary for the restoration of commercial activity on the river; and modern river terminals, constructed by the combined initiative of State and municipality, with a municipally-owned belt railroad, are to take their place. The passing of the gangplank is to be marked by the construction of concrete walls, against which the craft may be moored for convenient loading and unloading by huge cranes.

Instead of the droll chants of the indolent negro roustabouts, as they toil with the freight, we should hear the creak of machinery and the rattle of chains as the loads are hoisted from boat to warehouse. The old conditions along the levee must give way to immense warehouses and the mechanical handling of freight, if river transportation is to be revived.

The present excessive terminal cost often offsets the advantage of the lower river rate as compared with the higher rate by rail. Without an exception, there is no port in the Mississippi Valley at which freight can be transferred directly from car to boat, or *vice*

Mr. Bernhard. *versa,* except with a heavy switching arbitrary, or by an added labor cost equal to from 10 to 50 cents per ton.

For shipments originating, or consigned, locally, the drayage cost and the inconvenience of the present landings are such that each alone is sufficient to divert to rail routings a large volume of freight which otherwise would move by water at a distinct saving to the shipper, even under the other adverse conditions.

Not only in quays and facilities for freight exchange between rail and water, or in loading or unloading devices, do these river terminals fall short, but even the most commonplace and every-day necessities have been denied to them in the United States in general, and to those in the Mississippi Valley in particular.

Decent streets or trolley-car connections, even the telephone and telegraph, are strange and rare improvements on the river banks. New Orleans, which is the best river terminal in the United States, can at least show sheds and a municipal belt railway (even if the latter is often expensive by being frequently far from the water). This city has only four trolley-car lines to a 9-mile harbor front, with an hour's walk between each; but what about all the other cities in the United States?

St. Louis has no trolley car within four blocks of the used river front, four blocks which are paved with large, irregular, cobble-stones, with a slope on which it is difficult to walk, impossible to climb with horses, and dangerous to descend. Land anywhere on the river front of St. Louis and attempt to telephone, telegraph, or buy anything except beer and whiskey, and one will notice the total absence of necessities for the serious-minded man on business bent. It is the same in St. Paul and all other large cities, and what is true of them is certainly true of all the smaller ones.

These high terminal expenses are a serious menace to water transportation with the old-fashioned stern or side-wheeler, and are even more unproportionate and out of all reason where modern craft are used. The 1 000-ton, self-propelled barge that went to St. Paul from New Orleans in August, 1914, brought this forcibly to light. The most serious handicaps this barge encountered were terminals; in almost every instance it cost more to load and unload the cargo than to carry it to the place of destination.

The New Orleans freight was received, watched, and loaded, including the receiving clerk's time, for a total of 26½ cents per ton. The time consumed for loading was 5 hours, or 46½ tons per hour, during which time the barge expenses increased at the rate of $3.86 per hour, or a total of $19.30.

At Jeffries, La., 554.6 tons of lumber were received, being loaded and unloaded by the shipper. The loading took from Thursday at 3 P. M. until Monday at 10 A. M., working night and day and Sunday,

Mr. Bernhard.

a total of 91 hours, or 6.1 tons per hour. The loading cost the shipper 27 cents per ton. Loading the lumber on the barge cost, in time loss, 91 hours, or $351.26. The entire cost of loading and unloading, to the shipper, was $245.72, and this, together with the time loss to the barge, expressed in dollars and cents, makes a grand total of $807.35, as compared with the transportation of the entire lumber cargo of 554.6 tons for $998.37, or $1.80 per ton to carry the lumber 1 000 miles, and $1.46 to carry it to and from the barge.

With any proper semblance of an opportunity, this lumber should have been loaded on the barge in 12 hours, causing a saving in this item alone of 79 hours, or $305, and it is safe to say that what now cost $807.35 to load and unload should not have cost more than $210, under fair conditions, permitting an increase or decrease in rate of 60 per cent.

At Hannibal, Mo., 200 tons of cement were loaded, at a total cost to the shipper of $73, or 36.5 cents per ton. This was done in 7 hours, or at the rate of 28½ tons per hour. As the best "terminal facilities" were found here, a short description follows, the writer's criticisms being such as are justified by usual and average conditions in Europe.

Hannibal is on the Mississippi, has a population of about 14 000, and has a fairly flat and low bank. The city seems to be prosperous, and its streets are well paved and clean. The railroad station is practically on the river. The main line and some switch tracks of the Burlington Railroad are near the "river terminal", being about 80 ft. from the water's edge. This is partly due to the fact that the river bank does not run in a straight line, but is irregular. From the switch track nearest to the water, the bank has an easy slope to the river. The bank is paved with irregular cobble-stones on which it is very difficult to walk, so that there is little wonder that the laborers who loaded the cement, and crossed this path every minute for 7 hours, were far from efficient.

The bank is approximately 4 ft. above the water level, the side having an inclination of about 15° from the vertical. This bank is made up of stone blocks which apparently were meant to be thrown neatly on top of each other and thus form a bank, but the work had been poorly done, for some of the stones were lying in the river in front of the wharf, and one could see the rock blocks on the bottom, close to the bank, so that any barge coming near the bank and being gradually loaded, would settle down on these sharp blocks, to the great detriment of the barge bottom, and preventing a quick departure after the boat was loaded.

Just at what would be called the head of this "river terminal", is the wharf shed of the Streckfus Line. This projects into the river about 25 or 30 ft., and in the angle formed between this wharf-house and the quay, there are some piles which are now rotten. To the south

Mr. Bernhard.

of this bank, about 150 ft. from the Streckfus wharf, there is another wooden shed, but on the bank, and at that point the river terminal practically stops. A number of small and very frail motor craft and rowboats are anchored near-by, the outermost one about 30 ft. from the river bank.

The writer desires to state that he is not exaggerating, but is really describing what he believes was the best landing place on the Mississippi River at that date (August, 1914), with the exception of New Orleans, and equal to those at St. Paul or St. Louis.

On this bank the barge was to receive 200 tons of cement, at a rate of 50 cents per ton less than that by rail—or at two-thirds of the rail rate. It might be stated here that, besides the loss in time to the barge, the shipper, who saved $100 on his 200 tons by freight rate differences, paid $103 for loading and unloading, causing in the final transaction a loss to him of $3, notwithstanding the fact that the rail rate for a distance of 400 miles was reduced 33% by the barge.

Owing to the slope of the wall and the visible rock at the bottom of the river, the barge was lying about 3 ft. out from the bank; luckily, her bow happened to be at just about the same height as the river bank. The Atlas Cement Company, which was to load the cement, had arranged everything to the best of its ability. The cement was in eight box cars on the side tracks. To bridge the 3-ft. gap between the barge and the shore, a temporary platform was constructed.

The contract of transportation made with the Atlas Cement Company stipulated that the barge would carry no terminal facilities, or any facilities whatsoever for the handling of freight (not even a gang-plank) and that the loading and unloading was to be at the expense of shipper and consignee. The only gang-plank on the barge was a 2 by 10-in. board for the crew.

The cement was taken from the cars and placed on horse-drawn dump-carts, which were then drawn across the 80 ft. of bank to the platform, where they were dumped; there the bags of cement were loaded on hand trucks and rolled on deck to the place provided. This method of loading (at the best terminal on the Mississippi River except New Orleans) was the cheapest, and occupied the least time.

While the barge was being loaded, a deck-hand was kept busy sounding for rocks, in order to prevent the barge from sticking on one of them.

A switching engine was kept in waiting to move the box cars up to the place where they could be unloaded. The loading was done at the rate of 28 tons per hour. The wages of the engine driver and fireman amounted to, say, 84 cents per hour; or, only on their account, the cost of loading increased 3 cents per ton; or, on the tonnage moved

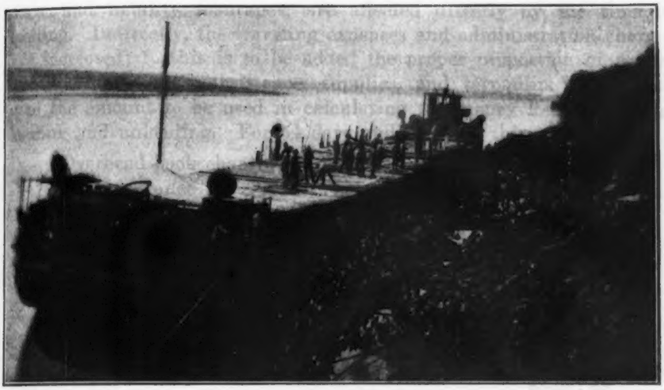

FIG. 3.—LOADING LUMBER AT JEFFRIES, LA. SPEED OF LOADING, 6.1 TONS PER HOUR. COST, 27 CENTS PER TON. COST OF BARGE PER HOUR DURING LOADING, $3.86.

FIG. 4.—HANNIBAL, MO., THE BEST TERMINAL ON THE UPPER MISSISSIPPI RIVER, WITH THE EXCEPTION OF DAVENPORT, IOWA, WHERE THE NEW RETAINING WALL IS NOT QUITE COMPLETED.

Fig. 3.—[illegible]

Fig. 4.—[illegible]

in this instance alone, their idle time would have paid interest for the entire year on $100 worth of terminal equipment. Mr. Bernhard.

Of the overhead charges of the barge, the book charges, hull insurance, and liability insurance, are affected directly by the time of loading. Indirectly, the traveling expenses and administration charges are increased; to this is to be added the proper proportion of wages, and fuel for light, subsistence, supplies, and tarpaulin hire, which give the amount to be used in calculating the money loss during the loading and unloading. For 54 days and 8 hours, this gives:

Overhead book charge	$819.18
Hull insurance	221.68
Liability insurance	25.13
Wages	2 244.13
Traveling and administration expenses	1 017.52
Light	29.16
Subsistence	400.00
Deck supplies	270.72
	$5 027.52

or $3.86 per hour.

The total cost of loading and unloading the barge on this trip was $1 733.12, or 63% of the gross freight receipts.

As a final word about the problem of inland navigation, the quarantine regulations which have to be contended with are worthy of a few remarks. Should there be bubonic plague in New Orleans, and should a vessel leave for foreign harbors with a health and inspection certificate from the Federal Health Officers, then such vessel will see such certificate honored wherever it goes; but should the authorities in the first harbor it touches desire to have it go through certain formalities, then at least all subsequent harbors will not further bother the vessel regarding its stay in New Orleans; but should a craft turn its prow up stream, and dare to remain within the jurisdiction of the United States officials, then any city or State health officer, regardless of any reasons to the contrary, and indifferent as to the number of former inspections and fumigations, may, on the boat's arrival within his "sphere of influence", stop, inspect, and fumigate to his heart's content; and so it came about that this steel barge, which was in New Orleans only 6 hours during the daytime, while the loading was in progress, and had been officially declared "absolutely rat-proof", was frequently detained by the various health officials along the route until the very end of its journey. At St. Paul, 1 921 miles from New Orleans, 35 days after its departure, it received its last health inspection, although it had lain 5 days off Jeffries, 3 days at St. Louis, and shorter periods at various other places, had been officially

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fumigated, and had had hundreds of visitors. The writer firmly believes that it escaped more serious delay because many officials were puzzled as to how to fumigate such an air-tight, steel structure, which had an open engine-room and pilot-house, carried its freight out in the open on deck, had its hold sealed with water-tight bulkheads, and had a deck without hatches. It is possible that some of these inspections were due more to curiosity about the construction of the barge than to sanitary reasons.

The barge left on July 6th, at 4 P. M., and returned on August 29th, at 2 P. M., a total of 54 days and 8 hours, including loading and unloading; and during that time covered 3 843 miles, spending 240 hours and 45 min. in loading and unloading 1 182 tons. She lost 8 hours and 4 min. on account of engine trouble, was grounded for 174 hours and 10 min., or waiting for the pilots to ascertain the location of the channel, notwithstanding the fact that at no time was a less depth than 6 ft. found.

The barge carried coffee, rice, sugar, cement, molasses, lumber, flour, paper, and chemicals. Out of the total of 1 921 000 up-stream ton-miles capacity, it delivered 1 066 986 ton-miles, and of the 1 921 000 down-stream ton-miles, it carried 183 181 ton-miles, or a total for the round trip of 32% of the barge's capacity. The total gross receipts were \$2 762.57, or 2.21 mills per ton-mile; the same freight moved by rail would have cost \$4 932.98; thus the water rate caused a saving of \$2 170.41, or 45% of the rail rate, which was an average of 5.98 mills per ton-mile as against 2.21 mills per ton-mile by water or 4 mills per ton-mile over the same mileage (the rail distance being shorter than the water distance).

The total expenses of the barge were high, and some items were out of all proportion on account of the lack of organization and other consequences of this "isolated" trip (guests, extra crew, etc.). For instance, the extra pilots, needed because the crew was totally unfamiliar with the route, received for this single trip in wages \$1 014.13, or \$18.67 per day, an item entirely in addition to the usual pay-roll.

Heavy expenses, incurred through slow loading, etc., to a large extent caused by the fact that this was an isolated trip, all made the expenses extraordinarily high. Under usual conditions, with a regular service and more boats than one to carry all the overhead and administration charges, the total expenses would have been less than \$3 500, but the total expenses for this 3 842-mile journey were \$5 740.42.

The overhead charges, including 6% interest on investment, 6% depreciation, ½% for repairs not covered by insurance, harbor dues, taxes, traveling expenses, and traffic manager (who, on account of this trip, was 6 weeks on the road), his wages, advertisements, photographs, traveling expenses of general manager, office and administration

expenses, telegrams, telephone, soliciting expenses, salary of clerks, etc., gives a total of $2 011.66. Mr. Bernhard.

The following are the details of the cost of the trip:

Fuel	$261.37
Deck supplies	270.72
Subsistence	400.00
Engine-room supplies	140.00
Wages of crew, including supercargo at $150 per month	1 230.00
Extra pilots	1 014.13
Hull, liability, workmen's compensation, cargo and freight insurance	412.34
Overhead charges	2 011.66
	$5 740.22

or a total of 1.65 mills per ton-mile of carrying capacity.

Had the barge been fully loaded both ways, the gross receipts would have been $8 491.00, at the rate of 2.21 mills per ton-mile (the average of her freight charges), yielding a profit of $2 750.78, even under these adverse conditions, on this single trip, and this, notwithstanding the fact, that, in mills per ton-mile, the freight rate of the barge was 37% of the rail rate, so that on the lumber alone the barge saved the shipper $1 300.52, making a rate of $998.65 as compared with a rail rate of $2 300.17, or $2 750.78 profit on a single trip, or 8½% on the investment.

The author argues that the United States is now in the second stage of waterway development. The writer might agree with Col. Harts if he would agree that we are now on the eve of the third stage and, as a matter of fact, going through the transformation stages.

Conferences are frequently held between the various interested parties, shippers, navigators, and officials. Only recently a conference held in St. Louis resulted in a permanent organization for the creation of modern terminals in the Mississippi Valley. Various organizations have been formed, one of the most active being the Upper Mississippi River Improvement Association.

Minneapolis is constructing a modern river terminal, well equipped with loading and unloading machinery, including a belt railway. St. Paul has similar work under consideration. Davenport has completed the first portion of her river terminal, Quincy has plans prepared, and St. Louis is making decided progress.

New Orleans is spending $3 000 000 on river cotton warehouses, and the State of Louisiana has recently passed a constitutional amendment commanding the Dock Board to dig or acquire a canal from the Mississippi River to Lake Pontchartrain within the City limits, the canal

Mr. Bernhard. to be used for river boats and the shores to be dedicated to private ownership, with warehouses suitable for river traffic. The writer believes that this canal will cost not less than \$3 000,000. The Panama Canal has brought the two coasts together so that the Middle West can no longer compete with the Atlantic Coast at the Pacific Coast or with the Pacific Coast at the Atlantic Coast. A clear illustration may be given by citing a paper published by the Corn Exchange National Bank of Philadelphia on January 5th, 1915, in which it is pointed out that, since the opening of the Panama Canal, general cargoes are being received at the Atlantic Coast from the Pacific Coast and transhipped by rail throughout fifteen different States, including Kentucky, Tennessee, Indiana, Missouri, Illinois, Michigan, and Wisconsin. This means that the Atlantic Coast warehouses and wholesale jobbers can undersell the Middle West houses in their own territory, and the only recourse the Middle West houses have is to establish warehouses at the Atlantic Coast (which means becoming Atlantic firms), or to secure cheaper transportation to and from tide-waters than their rail rate is at present to the Atlantic or Gulf Coasts.

The writer has another reason for being confident that we are now approaching what Col. Harts calls the "third stage". During the past 6 months the writer has received more than 300 letters from substantial firms urging river transportation as a solution of their problems. In one city alone 181 firms have signed a binding pledge to ship all their freight by water provided the rates are less than those by rail and the freight moves faster than the average by rail. Many cities and commercial organizations have made public pledges supporting water transportation. One instance may be given: Hannibal, Mo., on March 18th, 1915, made the following pledge:

"We, the Hannibal Commercial Club, fully realizing the great necessity and advantages of an economical and efficient navigation upon the Mississippi River, hereby pledge ourselves on behalf of our members to patronize a Mississippi Navigation Company through our members to the extent of 10 000 tons of freight outbound and 10 000 tons of freight inbound to Hannibal to be offered during the open season for navigation.

"These 10 000 tons of freight every year to be evenly distributed from April 15th to October 15th, providing that such navigation company that wishes to avail itself of this pledge maintains, at least, two sailings a week to and from Hannibal and eventually not less than 3 sailings a week; makes a rate of not more than 75% of the current rail rate, which rate is to cover transportation from warehouse at point of origin to warehouse at place of destination, or to the bottom of the ocean carrier, and to absorb all the switching and transferring charges, provided the place of origin and place of destination are on the rail track, in lack of which the rate is to apply only from the dock or to the dock, as the case might be; provided further that such a navigation company include in its rate a blanket insurance against all

damages and losses of whatsoever nature during the time that the cargo is in the possession of the navigation company. Mr. Bernhard.

"Provided further that such navigation company will only be considered to act as agent for the shipper or consignee where the freight is moved on through bills of lading over other carriers, and provided further that such navigation company guarantees that the average time of transportation of cargo will not be less than an average of 100 miles per 24 hours from warehouse to warehouse."

Now, the question is, can water transportation be cheaper and faster than rail transportation? The last contention will be discussed first. It is not generally known, but nevertheless it is a fact, that with the exception of some through freights, transportation by water is faster than by rail. Freight from New York to New Orleans by water is delivered in 5 days. What time would be required by rail? The many carloads shipped from New York to the writer in Alabama rarely took less than 10 days, and more frequently 20 days. A barge entirely inadaptable for Mississippi River service, having only 150 h.p. for 1 000 tons carrying capacity, and spending 18 days for loading and unloading and other delays along the river, on account of the poor terminals and channel lights, averaged on its round trip 3 miles per hour, inclusive of all delay, which, however, is more than 72 miles per day. What railroads to-day can show such a speed in their average shipments?

The writer does not want to use in this argument the oft-repeated statistics from the Interstate Commerce Commission, "that the average movement of a freight car in the United States is only 25 miles per 24 hours" because the Commission, in reaching this figure, has taken into consideration also the days that a car stands empty in the yards; but he affirms that no railroad in the United States can show an average movement of all its freight of more than 75 miles per 24 hours; and yet, with a proper system, it is very feasible to maintain on any water route an average speed of freight of 150 miles per 24 hours.

Water shipments—even with the present-day obsolete steamers on the Mississippi River and its branches—are everywhere faster than the average rail movement, which, after all, is natural, considering the yard delays of the railroads. Any one in doubt about these statements is cordially invited to make a trip with freight trains.

The writer refers now to carload shipments, and a considerably stronger story is told if shipment is made in less than carloads.

Many water transportation companies with which the writer has been identified have promised faster delivery than the railroads, except, of course, where they had to compete against milk or fruit trains—so-called fast or through freight train service.

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For the foregoing reasons, the writer reaches opposite conclusions, when the author states that "the usefulness of interior rivers as lines of communication has become much less important". The great importance of these lines of communication is just about dawning on the public. This, of course, is of value only if it can be shown that water transportation can be made cheaper than railroad transportation. That this is the case is quickly proved.

Beginning with ocean rates: From the Pacific Coast through the Panama Canal to the Atlantic Coast, inclusive of canal tolls, the average rate is $6 per ton, and the time of transport is 15 days. The average rail rate from coast to coast, in a more direct line, not paying canal tolls, is $22 per ton.

On the Lakes: Nowhere in the world are coal and ore moved more cheaply than on the Great Lakes, where the average charge is 0.4 mill per ton-mile. Compare this with the ideal rate held up by the author when he states that "some of the coal roads having easy grades and flat curves boast that their tonnage cost has been cut to 2.3 mills per ton-mile", or approximately six times higher than the Lake charges.

As to general package freight on the Great Lakes, this would still be handled at a great advantage over the rail rate were it not for the fact that water competition has been throttled by the railroads. To prove this the following is quoted from the *Chicago Daily News* of June 13th, 1914:

"On several occasions the *Daily News* has called attention to the throttling of water traffic between the Atlantic seaboard and western lake ports. The Chicago Association of Commerce has been active in trying to protect the interests of shippers by keeping open the water route between this City and the East. Through its traffic expert, Henry C. Barlow, the Association of Commerce has made vigorous protests before the Interstate Commerce Commission against any further advance in lake and rail rates as a part of the present rate raising move of the railroads.

"New York City merchants are now beginning to wake up to the importance of the issues involved, though they are not meeting the situation with as practical suggestions as are offered by Chicago. In a recent report the Merchants Association of New York calls attention to the facts brought out by the Committee on Merchant Marine and Fisheries of the National House of Representatives, in that rail and lake, and canal and lake rates have been advancing in recent years. while the all-rail rates have remained constant, with the result that shippers have found the water route less attractive." (This is shown in Table 4.)

"The recommendation of the Merchants Association of New York, is that the Interstate Commerce Commission requires the railroads to surrender all their boat holdings and leave the ownership and operation of boat lines on the Great Lakes to concerns independent of this recommendation. It fears the principal effect of it would be to

diminish service. The *Daily News* has heretofore questioned the wisdom of this recommendation. Mr. Bernhard.

TABLE 4.—FREIGHT RATES FROM NEW YORK TO CHICAGO.

Year.	Rate.	Classes. 1	2	3	4	5	6
1887....	All rail...........................	75	65	50	35	30	25
1887....	Rail and lake..........................	54	47	37	27	23	20
	Difference..........................	21	18	13	8	7	5
1914....	All rail...........................	75	65	50	35	30	25
1914....	Rail and lake..........................	62	54	41	30	25	21
	Difference..........................	13	11	9	5	5	4
1892....	All rail...........................	75	65	50	35	30	25
1892....	Canal and lake..........................	30	25	20	18	16	14
1913....	Canal and lake..........................	42	36	29	23	21	18
1913....	All rail...........................	75	65	50	35	30	25
1914....	No canal and lake rates.						

"The railroads now own all the boats in regular service between Buffalo and western lake ports. Driving those boats out of commission would do the public no good unless assurance could be given that others would be operated in their places. Independent lines do not develop quickly. The important thing is co-operation with the railroads so as to give satisfactory and cheap through service. Without that co-operation, independent boat lines would not help the public much.

"The Chicago Association of Commerce made a more practical recommendation when it asked the Interstate Commerce Commission not to permit further advances in lake and rail rates between western ports and the Atlantic seaboard. The important thing is to establish rail and water rates that should be substantially lower than the all-rail rates, and thus afford to points on the Great Lakes the advantages to which their location on the water naturally entitles them."

As to rates on canals: Take the old Erie Canal as an example—a canal completed in the first quarter of last century, with practically no improvements since its construction, and which held its own until the latter years of the Seventies—and, to-day, with the old canal fallen into disuse, the average rail rate between Buffalo and New York paralleling this canal is \$1.96, or $4\frac{1}{2}$ mills, per ton-mile, while the average cost in the United States is $7\frac{1}{2}$ mills per ton-mile. Seventy per cent. of the population of New York State is within 2 miles of this canal; 90% within 9 miles. This old canal is hardly a ditch, permissible only to boats with a beam not wider than 18 ft. and a length of 100 ft., entirely thrown into discard and not maintained in the last years on account of the new canal, yet to-day freight is moving through it, with the same old methods of a century ago, at a charge of 3 mills per ton-mile. Iron ore is to-day moved from Port Henry.

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Lake Champlain, through the Champlain branch of the Barge Canal and the Hudson River, to Kill van Kull, a distance of 260 miles, at a total charge of 80 cents per ton. So well is this canal thought of, that New York State is expending \$130 000 000 for a new canal constructed for more modern requirements.

Through Lake Borgne Canal—8 miles in length, widened to 100 ft. and deepened to 9 ft. by the writer—freight has been moved at a charge of 0.4 mill per ton-mile.

The New York State Barge Canal will reduce the cost of water transportation between Buffalo and New York considerably more, and reference is made to an article* by the writer in which in detailed figures it is shown that the cost of transportation between New York and Buffalo can be reduced to 26 cents per ton for the basic ton, that is, a ton of 2 000 lb. and occupying a space of 40 cu. ft. Plans are now being carried out as suggested in a report made two years ago by the writer to a well-known steel corporation, by which it is expected to move iron ore through the canal at a cost of ½ mill per ton-mile.

Can boats be built that will give as low rates on the rivers? Most decidedly, yes. The writer knows of no more difficult route in the United States than that from Tuscaloosa, Ala., to New Orleans, La. A description of the barges used on this route, with details of costs, etc., may be found in a recent technical publication,† to which the reader is referred.

To-day these barges are moving coal at a cost of ¾ mill per ton-mile over this route and returning empty. Just such a barge, entirely unfit for the Mississippi service, was used by the writer in August, 1914, in the trip from New Orleans to St. Paul and back.

To illustrate how cheap and profitable inland navigation in general, and Mississippi navigation in particular, can be, the writer gives some figures which he compiled recently for a group of financiers, who, on the strength of them, have agreed to furnish the needed capital of \$5 000 000 for the Inland Navigation Company.

This company, by May, 1917, will place on the Lower Mississippi, between New Orleans and St. Louis, five barges (one reserve barge) each of 3 000 tons carrying capacity and an up-stream speed of 15 miles per hour, so that a schedule may be maintained of a 3 000-ton barge leaving New Orleans and St. Louis, every Tuesday, Thursday, and Saturday.

These barges will have a maximum draft of 7 ft., will carry 1 800 tons at 5 ft., their minimum draft, and will be propelled by four propellers. The power will be furnished by four gas-producer engines, with a total of 2 400 h.p. The barges, entirely of steel, 310 ft. long, 56 ft. beam, and 9 ft. depth, will have a weather-proof cargo box.

* *International Marine Engineering,* August, 1914.

† *Engineering News,* December 4th, 1913.

Each barge will be equipped with wireless apparatus, motor launch, fifteen water-tight bulkheads, three 9 000-c.p. searchlights, distant anchor placer (new patent), and many important factors of economy. Each will be manned by a captain, three mate-pilots, one chief engineer, three assistant engineers, two oilers, two quartermasters, four deck-hands, two clerk wireless operators, one cook, and two stewards, or a total of twenty-one men, with a combined monthly pay-roll of $1 970. Mr. Bernhard.

The complete cost of each barge will be $312 000. The estimated cost of operation is based on an open season of 8½ months to St. Louis, 9 months to Cairo, 10 months to Memphis, and 12 months to Vicksburg. During the winter freight will be received as during the open season, but will be transferred to railroads where necessary, and in the winter it is anticipated that operations, at least for the first years, will take place at an actual loss.

The cost of operation of the barge is roughly as follows: 4% for marine, fire, public (property and life) liability, and cargo insurance, 6% interest on investment, 1% for repairs not covered by insurance, and 7% depreciation (12 years life), a total of 18% of $312 000, or $56 160 per year. For an every-other-day schedule, all fixed charges, wages, crew's insurance, and subsistence are to be increased by 25% to cover the reserve barge. Thus the total fixed charges are $280 800 per year. The working capital may be computed at the rate of $48 000 per barge, or $240 000; this makes a total needed capital of $1 800 000 for five barges. To this should be added auxiliaries, such as fast and powerful inspection tugs, each costing approximately $45,000, three of which are needed; two fast inspection launches for the management, at a total cost of $15 000, and a number of smaller launches stationed so that any stretch in the river may be reached within 9 hours by any of these tugs, or launches. There will be wireless stations at Memphis, St. Louis, and New Orleans, each having sufficient power to send messages a distance of at least 300 miles, the total cost of these stations to be $40 000. This, with $10 000 for office equipment and extras, makes $200 000 for extra equipment and shore installation, or a total of $2 000 000; 20% over this capital as profit, must, at least as a conservative estimate, be assured to justify capital to invest with safety in such a new enterprise to such an extent, which is $400 000 per year, or $100 000 per working barge, or $170 200 including fixed charges.

In theory the barge will make the trip from New Orleans to St. Louis in 80 hours running time, but in practice, when fog, wind, engine troubles, and grounding play an important part, an up-stream trip of 4½ days (which gives 35% extra time) should be estimated. As explained later, the loading and unloading at each terminal should not take more than 12 hours, so that the round trip may safely

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be based on 9 days, estimating a speed for the down-stream trip at an average of $17\frac{1}{2}$ miles per hour, with 22 miles in theory. On this basis the five barges will make 187 trips per year, so that the fixed charges with the profits amount to a total of \$3 640.60 per trip.

The crew's wages are estimated on a full year's pay of five crews for four barges, with 300 effective working days.

The wages per month are:

Captain (also pilot)	\$200
3 Mates (also pilots) combined salary, \$160, \$140, and \$120	420
2 Quartermasters	120
4 Deck-hands	180
Chief Engineer	200
3 Assistant Engineers, \$160, \$140, and \$120	420
2 Oilers	120
2 Clerk wireless operators	200
1 Cook	40
2 Stewards	70
Total per month	\$1 970

Thus the total wages are \$1 970 × 12 × 5 = \$118 200. The workmen's compensation and employers' liability insurance amount to \$1.10 per \$100 of pay-roll, or \$1 300.20 per year. Thus, the yearly charge for labor is \$119 500, or \$639 per trip. The subsistence, at 50 cents per head per day, 21 men per crew, and 5 crews per 365 days, is \$19 162.50 per year, or \$102.47 per trip.

The fuel, estimating a total of 200 running hours per round trip, burning breeze coal, with 2 400-h.p. engines, amounts to \$840 per trip. For each trip the supplies for deck and engine, including lubricating oil, will cost \$400, and the extras and contingencies \$250, so that each trip will cost:

Fixed charges, including interest and profits	\$3 640.60
Wages, including insurance	639.00
Subsistence	102.47
Fuel	840.00
Supplies	400.00
Extras, at 3%	291.00
Total	\$5 913.07

or, in round numbers, \$6 000.

With the barge fully loaded on each trip, freight could be moved from New Orleans to St. Louis for \$1 per ton, which would give a yearly dividend of 26% and establish a 100% sinking fund within 12

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years. In this the following factors make various changes: the protection against the yearly loss through part-rail movement in winter, the loading and unloading charges, switching charges, administration, soliciting, advertising, and the fact that the barge might not always be loaded to full capacity.

Each barge has a total yearly capacity of 224 400 tons each way. It seems safe to estimate that the barges will soon be loaded to 80% of their capacity, or 4 800 tons per round trip, or $1.25 per ton from New Orleans to St. Louis.

The barges will make 140 trips to St. Louis between April and December, 8 trips to Cairo during the late spring and late fall. 12 trips to Memphis in early spring and early winter, and 27 trips to Vicksburg in middle winter.

To St. Paul, for which services 1 000-ton barges with four propellers and with a maximum draft of 4½ ft. will be used, it is estimated that there will be an open season of 7½ months.

During the winter freight will be transferred to railroads at Cairo, Memphis, and Vicksburg.

The rail rates are such that during the winter (until these rates are adjusted by the Interstate Commerce Commission) the company should be prepared to stand an estimated loss of $180 000, because, in many cases, the rail rate from Vicksburg, Memphis, or Cairo, to St. Louis, is more than the water rate from New Orleans to St. Louis. This $180 000 merely represents the final figure after having for one month moved freight to Cairo, transferred it to the railroads and paid the rail rate from there to destination, and done the same thing for one month at Memphis, and two months at Vicksburg. This makes a total additional charge of 17½ cents per ton over the entire year. The administration, soliciting, and advertising is estimated to be $70 000 per year, or about 7½ cents per ton. Thus, outside of loading and unloading, or switching charges, the total rate per ton from New Orleans to St. Louis should be $1.50 per ton, and commands an investment of $2 000 000.

There will also be needed some small repair stations along the route, because of the total absence of efficient shipyards; a floating dry dock at St. Louis, and a small sectional dock at Memphis. This entire equipment may be secured for $300 000; to this may be added about $100 000 for cost of organizing expenses and $50 000 for various shore equipments, so that the total capital necessary for this company is $2 500 000, of which $500 000 should be incorporated as a separate company—a ship-building company. Detailed estimates have satisfied the writer that such a ship-building company will be sufficiently profitable through outside work to be self-sustaining, and that it will

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be proper not to put this $500 000 burden on the earning power of the barges.

The foregoing gives a fairly good idea of the operating conditions of the new Mississippi River Transportation Company, as far as the water transport is concerned.

Relative to the shore expenses, there remains the cost of loading and unloading, and the switching charges. The cost of loading and unloading includes only the removal of freight to and from the barge. The problem is to do this as economically as possible, with the greatest safety to the cargo, and with the greatest dispatch to the boat. It should not be forgotten that the value of the barge is $32.22 per hour, thus every hour's delay is equal to a tax of 1.34 cents per ton, so that economical transport with the greatest dispatch is of the utmost importance, because 12 hours for loading and unloading purposes means 16 cents per ton of the freight expenses, and somebody pays the bill.

To accomplish this quick dispatch, it is the intent to make stops only at cities where there are proper modern terminal facilities, or which, either as a community or through individual shippers, have signed for sufficient freight to and from, to justify the company in establishing its own river terminals at such cities.

The loading and unloading, and in fact all shore charges, are to be taken care of by a separate company, which will be known as the River Terminal Company. This company will look after the loading and unloading of barges, the receipt of freight from shippers, and delivery to consignees.

In order to give municipal enterprise ample opportunity to build its own terminals, without finding the River Terminal Company in its way at any time, and in order to keep before the public the fact that this River Terminal Company is only a makeshift necessitated by the very poor river terminal conditions, the company will be ready at any time, if so desired, to exchange local property for municipal terminal bonds.

In order to remain independent, it is proposed to build some of these terminals as floating terminals, so that they may be removed. The deck of such a terminal can always be at the same height as that of the barge deck; and the terminal is to be connected by tracks, running over the roof of the terminal barge cargo box, over a hinged apron, to the belt railway, if there is such, or otherwise to the various railroad tracks.

This floating terminal collects and disburses the freight, and care will be taken that it is ready for loading on the arrival of the barge, and also that preparations are made to have the loading and unloading proceed simultaneously.

Such a terminal will always have a little more than twice the capacity of the barge which it serves; in other words, for St. Louis

(not of course the smaller places), the transfer barge will have a capacity of 6 100 tons. Mr. Bernhard.

These terminals will be equipped with wireless apparatus, two 10-ton traveling cranes with 40-ft. booms, and also telphers, conveyors, and electric trucks.

The main deck of the terminal barge will connect directly with the street by a roadway 40 ft. wide running through the center of the barge from end to end. At each end of the roadway there will be an automatic track scale where the driver will receive a scale ticket giving the total weight of wagon and freight. On leaving he will pass over another scale at the other end of the barge, and on the same ticket, the weight of the wagon will be recorded and subtracted automatically from the first weight, thus showing the total weight of the freight left behind or received while on the transfer barge. The freight will be unloaded, assorted, inspected, rated, stacked, and the results concisely marked on the weight ticket after the driver has passed the last scale. He will turn his ticket in at the office at the end of the barge and receive his bills of lading in return.

The height of the transfer or terminal barge will be regulated by ballast water, pumped out or let in, as necessary, so that the transfer and transportation barges will be always at the same height during the loading and unloading. The transfer barge will also be equipped with electric light, and a lunch counter, which will facilitate day and night loading. As a barge will come and go every other day, and freight will be received or delivered every day, the crew of the barge will find steady employment. By having regular employment, these 'longshoremen will become efficient in receiving, loading, and unloading freight from the barges, or transferring it to and from the railroad cars. The company will load and unload cars for local distribution, so that freight can be delivered to and from the warehouses in river towns or to the railroads for delivery in the interior.

Three belt conveyors for general bagged, boxed, or barreled materials—such as coffee, rice, sugar, flour, cement, etc.—and four telpher trains (four cars in each train) for smaller freight, like canned goods, will enable 75 men to handle this general merchandise at the rate of 500 tons per hour, thus completely unloading a barge in 6 hours. The loading can commence practically within an hour after the unloading is started, so that the total time required for loading and unloading should not exceed 7 hours. If it really requires 10 hours to unload and load the entire cargo, the time is still well within the 12-hour limit set for this work.

It is assumed that the total crew on shore will consist of 75 laborers, 5 mechanics and electricians, 2 weigh-masters, 2 wireless operators, 8 clerks, 10 receiving clerks, 1 dock captain, 1 supply agent, 2 policemen, 2 watchmen, 1 carpenter, 2 engineers, 3 messengers, 3

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cashiers, and 5 bookkeepers, all at a combined yearly pay-roll of $125 000, plus insurance, or about 14 cents per ton received or delivered. The cost of power is estimated at 1½ cents per ton received or delivered.

The transfer barge, complete with all its machinery and mechanical appliances, power, etc., will cost $300 000; and interest, depreciation, insurance, repairs not covered by insurance, and harbor dues, will amount to 15% per year, or 4½ cents per ton.

Thus, the total shore expenses, outside of switching charges or car hire at each end, will be 20 cents per ton; estimating the profits at 20% of the cost of the terminal barge, the charge would be 25 cents per ton.

The switching charges will be from $2 to $10 per car, or from 5 to 30 cents per ton. The company's charges to the shipper will always read about like this:

"Switching charges from your warehouse in your town to river, plus loading charges in your city, plus river transportation, plus loading at consignee's town, plus switching charges in his town, gives a total of ...."

Thus, the total charges from New Orleans to St. Louis are: 10 cents for switching charge and car rental, 25 cents for loading, $1.50 for transportation, 25 cents for unloading, and 20 cents for car hire and switching charges in St. Louis, or $2.30 per ton. This charge will be reduced to $1.92 as soon as the Interstate Commerce Commission has readjusted (or rather corrected) the present switching and rate arbitraries. This rate covers receiving in cars at warehouse and delivery at warehouse of consumer (during both winter and summer), within a guaranteed time of 8 days, except where rail movement is necessary, when the customary indefinite and greater time must be allowed, under full insurance against all damages; or at a rate of $2 when received and delivered at the river terminals, which charge will be reduced to $1.82 per ton when the rates are corrected; and if it is guaranteed that the barge will be loaded to full capacity, the charge will be reduced to $1.45 per ton. The latter should be the ultimate and total charge, and it is the aim of the Inland Navigation Company to be able to make a rate of 1.2 mills, water distance, or 2 mills per ton-mile, rail distance.

The barge line will be classified as a common carrier, and will be controlled by a bill of lading which is now being drawn up by expert lawyers. This bill of lading will contain some very novel clauses; for instance, it will have one clause protecting the cargo against all damage or loss, and another guaranteeing the time of delivery. It will be a through bill of lading, and the rate will be scientifically based, as shown below, so that any one knowing the rules may know what the freight charges will be for each commodity, in his case, without ever looking at the tariff sheets.

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Thus far the general cost of carrying the freight per ton was estimated, without taking into account the variation in the character of the freight carried, which, of course, is essential. The ton was considered as 40 cu. ft. bulk capacity, and not more than 20 ft. in length, width, or height; not fragile or too weak to have a load placed on top of it, 10 ft. in height and weighing not more than 50 lb. per cu. ft.; not subject to damage on account of weather conditions; to be contained in individual packages weighing not more than 3 tons; without offensive odor or susceptible to odors; deviation from any one of these conditions will alter the charge. Taking the freight rate of $1.50 (or 1.2 mills) per ton-mile as a basis: for each cubic foot that the freight occupies in excess of the 40 cu. ft. per ton, one-fortieth of this charge shall be added; for each fraction that the individual package is in excess of the 20-ft. length, width, and height, the freight increases proportionately. Commodities in individual packages, which, when dropped from a height of 10 ft. are subject to breakage, will have their freight cost increased at the rate of 0.02 mill for each inch this fall is shortened, so that for a commodity which might break at a drop of 1 in., 2.4 mills per ton-mile are added to the basic rate.

In addition, the freight will be subject to extra insurance on values greater than $100 per ton, so that, for each dollar of additional value, 2 mills will be added, regardless of distance; thus, if the commodity is worth $300 per ton, the additional cost for moving it will be 40 cents.

A material which is too light in construction to have any cargo safely placed on top of it will be charged as if it occupied a height of 10 ft. (the height of the cargo box). If any material has an odor (such as fertilizer) so that it might affect other commodities or attack quickly the odor of other commodities (such as flour), it will be subject to an increase of 0.2 mill per ton-mile. This is to protect against the extra cost of placing such cargo in special compartments. Cargo which is affected by temperature will have to pay for refrigerating or heating, as the case may be.

Cargo which is excessive in weight is charged with an increase of 0.003 mill per ton-mile for each pound more than 50 lb. per cu. ft. Individual packages which weigh more than 2 tons will be subject to a charge of 2½ mills per additional pound. Thus, a 10-ton package (transported from New Orleans to St. Louis) will cost $44.60.

Of course, it would hardly pay for the barge to stop at each landing, as was heretofore customary. This makes the following rule necessary: the barge will not stop at any landing for less than $50, regardless of the quantity of freight offered. However, one might keep loading the barge for an hour (the freight charges being at the usual rate), with this understanding: that after so much freight had been

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put on the barge that its cost of transport reached the $50 mark, any additional cargo put on board within the first hour would be free, up to $50. The same method will be true for the second hour, etc.; further, the rate will be subject to an increase according to the decrease in distance, so that the rate might be considered as a sliding scale of 5 mills per ton-mile for distances less than 10 miles; 4 mills per ton-mile for distances less than 100 and more than 10 miles; 3 mills per ton-mile for distances less than 150 or more than 100 miles; 2 mills per ton-mile for distances less than 200 and more than 150 miles; 1½ mills per ton-mile for distances less than 500 or more than 200 miles; 1.4 mills per ton-mile for distances less than 1 000 or more than 500 miles; 1.2 mills per ton-mile for distances less than 1 500 or more than 1 000 miles; 1.1 mills per ton-mile for distances less than 2 000 or more than 1 500 miles; with the understanding that a lower rate for longer distances should never result from this; in such cases the higher rate would always apply.

For freight consigned to interior points, the lowest rail rate from the river to the town will be added, provided such lowest rate is from a place where the company has terminal facilities, or where good facilities can be acquired.

Such freight as will take advantage of the clause of the bill of lading guaranteeing delivery in a specified time will be charged at the rate of 0.1 mill extra per ton-mile.

The foregoing rates are compared with railroad rates in Table 5.

It will be seen that a rate of 2 mills per ton-mile, water distance, including all expenses (such as delivery during the closed winter season, insurance, absorbing of switch and belt railroad charges, and the warehouse delivery) is profitable.

Water transportation in the United States has dwindled to nothing, on account of artificial conditions, but there is no reason on earth why our rivers should not be teeming with traffic; and there is every reason to suppose that at a date not far distant such traffic will start. The usefulness of our rivers is becoming more and more apparent to the public, and, at the present time, it is also a dire necessity that the streams be used. Foreign competition demands an early adjustment of the methods of interior transportation.

The writer is in hearty accord with Mr. Lavis when he states that our interior lines of communication must be developed to carry our products from one part of the country to another at the lowest possible cost per ton-mile. In our competition for foreign trade we should look on the whole United States as one large factory competing in these foreign markets with other factories (such as the English, German, and French), and it is essential for such competition that we reach a high stage of efficiency in our factory; that we put to use the potential value of our waterways. The non-use of these rivers and water routes

constitutes a voluntary penalty and handicap under which we are placing ourselves in favor of foreign competitors, and our cry in the new battle for foreign trade should be: "Back to the Waters". Mr. Bernhard.

TABLE 5.—COMPARISON OF FREIGHT RATES BETWEEN NEW ORLEANS AND ST. LOUIS.

Commodity.	Rail rate, per ton, for carload lots.	Rail rate, per ton, for less than carload lots.	Water rate, per ton.
Baking powder	$7.50	......	$3.20
Beef	5.00	......	4.10
Beer	4.20	$5.20	2.30
Bridge material	3.50	5.40	3.25
Butter	18.00	......	4.00
Canned goods	5.20	6.90	2.80
Cement	2.50	6.10	2.25
Chairs	19.50	......	6.00
Cocoa	13.50	......	5.40
Coffee	4.60	6.00	3.10
Condensed milk	4.20	8.70	3.15
Copper ingots	7.00	13.00	2.50
Copper castings	15.00	......	4.00
Dry goods	13.00	18.00	4.50
Flour	2.80	5.20	2.50
Furniture	8.50	11.50	5.80
Glassware (bottles)	10.30	13.50	3.70
Guano	61.00	73.20	8.00
Hardware	6.00	......	3.60
Liquors	8.00	......	3.10
Lumber	4.00	......	2.28
Molasses	4.10	......	3.00
Nails	4.40	......	2.80
Nitrate of soda	3.40	73.20	3.10
Paint	4.60	......	2.60
Paper	5.60	10.00	2.50
Plumbing material	6.00	......	3.40
Pickles	76.00	......	2.40
Refrigerators	10.00	......	2.90
Rice	4.80	......	2.40
Rubber	18.00	......	3.00
Salt	4.30	......	2.95
Shoes	18.00	......	6.00
Soap	4.60	......	2.40
Stoves	7.00	......	3.00
Sugar	3.40	4.80	2.70
Tin	10.00	15.00	3.80
Vehicles	5.00	......	4.00

The writer does not believe that we need to look forward to smaller expenditures on our waterways, but believes that saner methods of spending money on them are required. Instead of making yearly haphazard appropriations, each project in itself should be approved or condemned, and, if approved, authority should be given to carry out the work, a lump sum—based on the estimate of competent engineers—being set aside for the purpose, instead of as it is now done, on the installment plan, with frequent intervals of idleness on account of some filibuster or change in politics. How frequently has it happened that important work has been stopped, costly equipment set idle, and splendid organizations dispersed, because Congress did not that year

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appropriate the necessary money to continue. What sane business man or contractor would attempt to carry out work on such a basis?

The author states:

"Work on these canals and highways requires many engineers. Up to 1824 the Military Academy at West Point was the only school in the country which trained men as engineers, and for many years thereafter it was the principal school of this kind."

For that very reason we are still employing as engineers for our waterways the army engineers of the Government, trained as army engineers and not as water or transportation engineers. It is a high tribute to the efficiency of these men that the work has been carried out in such a splendid way; but, is it right?

In reference to this, the writer calls attention to the Kingdom of the Netherlands, where the Government employs in a separate department—called the "State of Waterways" (literally translated)—a corps of engineers, only and solely trained as water and transportation engineers. These men have been sent the world over to construct waterways. The Suez Canal is to-day maintained by Hollanders; Galveston, Valparaiso, Rio de Janeiro, Buenos Aires, Bahia, Hong Kong, Santiago, Macassar, Macao, Santander, Bilbao, Hamburg, and many other harbors and waterways, prove their efficiency. Holland's dredges, floating equipment, levees, locks, and canals, are world famous.

The writer agrees with Mr. Lavis that our army engineers have given ample proof of their efficiency, but this is rather a tribute to their brains than to their training. Long ago there should have been in the United States a separate department (with a cabinet minister at its head) for waterways. The success of foreign waterways (as stated by Mr. Lavis) is largely due to the fact that the matter of deciding on the project for which public moneys shall be spent, has been left generally to a comparatively small body of men of training and experience in the particular class of work or problem involved, whereas in the United States this is usually decided by popular vote.

Notwithstanding what has been said about the splendid work performed by the army engineers, the writer claims that it is a decided error in judgment to have them pass upon the commercial value of a project presented for their consideration, and, as stated by Mr. Lavis, it is no wonder that "their advice is frequently ignored", as their training and experience do not make them proper judges of the commercial value of such projects. How frequently have they stated that certain needed links between waterways were of no commercial value because there was at that date no actual freight moving over that route?

The writer has in mind one particular case: the canal from Stillwater, Wis., to Lake Superior. What greater need of a link between

waterways could have been thought of in that State? Yet the engineer in charge stated that such a link would have no commercial value. Mr. Bernhard.

The author, in order to cite the importance of our waterways from a financial point of view, points with pride to the fact that the work now yearly requires $40 000 000, or more than eightfold what it required 40 years ago.

Table 6 gives an interesting comparison of how rivers and harbors have shared in Federal appropriations during the last 40 years.

TABLE 6.—FEDERAL APPROPRIATIONS FOR ARMY, NAVY, PENSIONS, AND RIVERS AND HARBORS, SINCE 1875.

Year.	Army.	Navy.	Pensions.	Rivers and Harbors.
1875..........	$27 788 500.00	$20 813 946 20	$29 980 000.00	$5 218 000.00
1876..........	27 938 830.00	17 001 006.40	30 000 000 00	6 643 517.50
1877..........	27 621 867.90	12 742 155.40	29 533 500.00	5 015 000.00
1878	25 812 500.00	13 541 024.40	28 533 000.00	....
1879..........	25 593 486.01	14 152 603.70	29 371 574.00	8 201 700.00
1880.....	26 797 300.00	14 029 968.95	56 233 200.00	7 846 600.00
1881..........	26 425 800.00	14 405 797.70	41 644 000.00	8 951 500.00
1882..........	26 687 800.00	14 566 037.55	68 282 306 68	11 441 300.00
1883..........	27 258 000.00	14 819 976.80	116 000 000.00	18 738 875.00
1884..........	24 681 250 00	15 894 434 23	86 575 000.00	....
1885..........	24 454 450.00	14 980 472.59	20 810 000.00	13 949 200.00
1886..........	24 014 052 50	15 070 837.95	60 000 000.00	....
1887..........	23 753 057.21	16 489 937.20	82 075 200.00	14 473 900.00
1888..........	23 724 718.69	25 767 348.19	82 152 500.00	....
1889	24 471 300.00	19 942 835 35	85 258 700.00	22 397 616.19
1890..........	24 316 615.73	21 692 510.27	89 758 700.00	....
1891..........	24 206 471.79	24 136 035.53	123 779 368.35	25 136 295.00
1892..........	24 613 529.19	32 541 654.78	164 550 858.34	2 951 200.00
1893..........	24 300 499.82	23 543 385.00	154 411 682.00	21 968 218.00
1894..........	24 225 639.78	22 104 061.38	180 681 074.85	14 166 153.00
1895..........	23 592 884.68	25 327 126.72	151 581 570.00	20 043 180.00
1896..........	23 252 608.09	29 416 245.31	141 381 570.00	11 462 115.00
1897..........	23 278 402.73	30 562 660.95	141 328 580.00	16 244 147.00
1898..........	23 129 344.30	33 003 234.19	141 263 880.00	20 832 412.91
1899..........	23 193 892.00	56 098 783.68	149 304 702.46	14 627 449 56
1900..........	80 430 204.06	48 099 969.58	145 233 830.00	25 110 038.94
1901..........	114 220 095.55	65 140 916.67	145 245 230.00	16 285 605.75
1902..........	115 734 049.10	78 101 791.00	145 245 230.00	7 046 623 00
1903..........	91 730 136.41	78 856 363.13	139 842 230.00	32 540 199.50
1904..........	77 888 752.83	81 876 791.43	139 847 600.00	20 233 150.00
1905..........	77 070 300.88	97 505 140.94	142 360 700.00	10 872 200.00
1906..........	70 396 631.64	100 336 679.94	142 750 100.00	28 726 007.41
1907..........	71 817 165.08	102 091 670.27	143 745 500.00	17 254 050.04
1908..........	78 634 582.75	98 958 507.50	147 143 000.00	43 500 813.00
1909..........	95 382 247.61	122 663 885.47	173 053 000.00	18 092 945.00
1910..........	101 195 883.34	136 935 199.05	160 908 000.00	29 190 264.00
1911..........	95 440 567.55	131 350 854.88	155 758 000.00	49 380 541.50
1912..........	93 374 755.97	126 478 338.24	156 182 000.00	30 883 419.00
1913..........	90 958 712.98	147 077 507.76	165 146 145.84	33 259 870.50
1914..........	94 416 143.51	162 097 167.53	180 800 000.00	47 868 894.00
Totals.........	$1 973 825 531.68	$2 120 214 833.29	$4 568 251 057.52	$680 552 501.01
Yearly average for 40 years.......	$49 345 638.29	$53 005 370.83	$114 206 276.44	$17 013 812.53

The total amount, $680 552 501, spent on rivers and harbors is only 4% of the total amount spent on railroads, and less by far than the

Mr. Bernhard. total value of land grants alone made to the railroads, amounting to 160 000 000 acres, larger than all that part of the United States east of the Ohio and north of Maryland; the total freight carried on these railroads in 1914 was 288 320 000 000 ton-miles; yet on the Great Lakes alone in that year the writer estimates that the freight was 30 000 000 000 ton-miles.

This grand total of money spent on our waterways is smaller than the grand total spent on the waterways of Holland, a country one-two hundred and fiftieth in area with one-seventeenth of the population.

Here the writer desires to point to what is apparently a discrepancy in the author's statement where he cites that Prussia spent, between 1813 and 1906, $129 000 000 for her waterways, and later he uses this in his comparison with Germany, as though this $129 000 000 spent by Prussia was all that the German Empire spent.

The United States has not spent enough by far for waterways, and much more will have to be expended.

Finally, the writer wishes to point out that practically not a dollar has been expended on our waterways without the approval of the army engineers. All work must first pass their close examination, as to feasibility and need, and no project stands much chance to pass both House and Senate if not approved by them. Some small works may have passed, but certainly not costing more in total than one-half of 1% of the money spent.

In comparing rail rates with water rates, one must also take into consideration the large land grants, franchises, rights of way, and bonuses given to the railroads, in many instances representing fabulous wealth, and certainly in their present-day value far in excess of the total sum spent by the United States up to date on its waterways and harbors.

Railroads often advance the argument given by Mr. Lavis, that, to estimate the actual cost of water transportation, the true expenses of maintenance, interest, and operation should be charged against the water traffic; but, granting that this is correct, which the writer is not at all inclined to do, would it not be equally just to demand of the railroads interest on the land grants and royalties on the minerals granted to them? If such argument held true, we should charge tolls on our streets and country roads; our waterways are just like them—free and open to all; any one, from a rowboat capitalist to a trust magnate, may use them, just like our streets and roads. Why only charge for the use of the streets? Why not also charge for underground use, for the rail tunnels in New York City, Baltimore, Washington, etc.? Such faulty arguments lead to ridiculous conclusions. Suppose we did not improve our rivers and just permitted them absolutely to pass away as useful channels of trade, then, of course, there would not be any charge to collect, and we would save

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the interest, depreciation, and operating expenses of our waterways; and does Mr. Lavis think that this would be very constructive to our prosperity and national wealth? If he does, the writer may cite for him a few instances where the river channels became useless and where rail rates jumped in leaps and bounds. If this argument held true, streets should be torn up in the cities and everybody forced to move by trolley car; busses should be forbidden by law; country roads should be made impassable; and it should become a criminal offence to walk instead of patronizing the railroads.

Often the argument is advanced that our railroads move freight at greatly less charges than those of Europe. Mr. Lavis says:

"It is probably true that, in the development of inland waterways, some of the European nations have attained a greater measure of success than we have, but in economic progress in the general realm of transportation, including that by railways, it is undeniable that we have far out-distanced the rest of the world."

He also says:

"In comparison with the rates of wages, the cost of freight transportation on the railroads of this country is only from one-third to one-quarter as much as in Germany."

Now, this general assertion is incorrect. It is granted that the official figures show a higher cost per ton-mile for the freight carried in Germany, England, or other European countries, than in the United States; but here is where statistics lie, because they are misrepresented.

The low-class commodity and bulk freight in Germany, France, Belgium, and Holland moves by water, and in England, where the inland water system is killed by the private ownership of railroads, it is chiefly hauled coastwise. If all such low commodities as move by water in these European countries, with their low rates, are eliminated from the figures for the United States, the general average cost of transportation would be considerably higher. In those territories of the United States which have no large tonnage of low-class, bulky freight commodities, the average per ton-mile is from 2 to 3 cents per ton, or considerably higher than the average in European countries.

To get at a fair comparison between rates in America, Germany, France, Belgium, Holland, and England, there should be omitted from the American figures all those pertaining to the movement of coal, ore, lumber, grain, oil, and steel. There would also have to be taken into consideration the fact that European railroads had no land grants, but very expensive rights of way, together with a vast number of terminals per mile or per ton carried.

Another phase which must be taken into consideration is that, on account of the density of population in Europe, the demand for smaller cars is strongly felt, so that carload shipments there average from 10

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to 15 tons, resulting again in the fact that the rolling stock per ton-mile is far greater than in the United States. Intershipment of freight between small communities has reached a greater percentage of efficiency because of these small cars than in the United States, where many good-sized shippers find that their requirements of space still fall short of the prescribed carload shipment, which means a higher rate and greater reduction in the speed of movement, as a natural consequence of mixed shipping in part carloads.

When all these facts are taken into consideration it will be clear that an empty comparison between the published European rail cost per ton-mile and the average cost in America is an unjust basis on which to assume that the rail cost of transportation is cheaper.

The author states:

"It is a wise policy, amply justified by experience, to have harbor facilities always a little in advance of the immediate necessities, and many of our seaports have responded admirably to efforts in this direction."

In reference to this statement it may be proper to remark that there are no efficient harbors in the United States; even the best and cheapest of them compare very unfavorably with European or South American ports, with the possible exception of England, more especially London—where the harbors are noted for their expensiveness. The second harbor in the United States, which is also the cheapest, namely, New Orleans, up to a recent date, could not show a single crane, and in general this harbor would be considered far from being a model of efficiency. Any one who has visited large modern foreign ports will remember their floating, stationary, and movable cranes, and other mechanical appliances for loading and unloading.

This being a discussion of rivers and railroads, the writer will restrict himself to these general statements, although a comparison of the cost of handling freight to and from steamers in the harbors of the world would be interesting. It would bring out many a fact of which we would feel ashamed, though the cheapest methods of loading and unloading coal and ore are on the Great Lakes where (thanks to private enterprise) they are handled with remarkable dispatch at a cost per ton never heard of in Europe.

The author says:

"A practical test, almost infallible in its application, that will show whether a waterway project can be economically considered for further improvement, is a progressively increasing commerce, and the measure of saving in cost of transportation will always be a guide as to the extent of work that is justified."

The writer cannot agree with this. Such a test cannot be the right one. Decrease of traffic on a water route (or on any other route, as

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far as that goes), does not necessarily indicate the need of the stopping and abandoning of all further improvements; it merely indicates that there are causes which handicap the use of such routes, and it becomes necessary to find out what these causes are, before further improvements are made or abandoned. After such investigation, it may be clear that the absence of these very improvements brought about the decline in traffic.

Mr. Lavis gives two arguments that influenced the people of the State of New York to authorize by vote the construction of the new canal: first, that by providing competition the canal would compel the railroad lines to keep freight rates down; and second, that the actual cost of transportation would be less by the canal than by railroad.

That the water routes have kept the railroads down in the past has been proved so often that this does not need to be thrashed out again. That such should not be the case the writer has frequently contended. He is convinced that waterways built with the sole idea of bringing down rail rates, not of using the routes, proves an error in judgment. A water route should be created only if wanted for its direct advantages, not because it becomes a club to beat down rail rates.

Mr. Lavis agrees that the actual cost of transportation by canal is less than by rail, provided the interest on the investment and the cost of administration and maintenance of the canal are ignored. It is clear that his statement, that such overhead charges are about 8.6 mills per ton-mile, is only based on an estimated tonnage that would move through that canal; thus, with every ton in excess of Mr. Lavis' figures, this overhead charge of 8.6 mills per ton-mile decreases.

The water distance between Buffalo and New York is approximately 500 miles, so that the overhead charge for this distance is 43 cents per ton. Now, it has been shown* that the cost of transportation could be 26 cents per basic ton from New York to Buffalo, which makes a total of 69 cents per ton. Compare this with $1.96, the average railroad charge between Buffalo and New York, and $1.09, the lowest rate. It is clear that with an efficient service on this canal, and a rate which includes 22% profit—is in fact 46% of the rail rate—a very much larger tonnage would move through the canal than that on which Mr. Lavis has based his figures. This in turn would automatically decrease this 43 cents overhead charge.

When Mr. Lavis states that the new canal cost considerably more per mile than the New York Central Railroad, it must be remembered that the capacity of the canal is many times larger than that of the railroad.

Mr. Lavis advances the argument that if the New York Central Railroad had been presented with, say, one-half the cost of the Barge

* *International Marine Engineering,* August, 1914.

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Canal, and one-half the estimated annual cost of administration and maintenance, it would have been glad to guarantee a freight rate of 2 mills per ton-mile on such items of bulk freight as may be expected to pass through the canal during the season when it is open. This sounds all right, theoretically, but it is very hard to agree as to what might move through the canal. In reports made by the writer to various individuals and corporations, and in his last report to one of the cities along the canal, he pointed out that undoubtedly a very large tonnage of general package freight and first and second-class freight would move through this canal; and numerous are the instances that could be cited where water transportation companies handle first-class freight and package freight in preference to the railroads.

Attention is invited to the Sacramento in California, the Trenton Transportation Company of Trenton, N. J., or to the various transportation companies in Florida where, for instance, the Howard Transportation Company handles fruit (especially citrus fruit) to the exclusion of the railroads, because it can forward it more rapidly and at a lower rate.

Mr. Lavis contends that in the United States the railroad, as an efficient and cheap means of transport, has made possible the development of its principal industries where the soil, climate, and other conditions were best suited to them. Far greater credit is due to the waterways. Even to-day, at the ebb of water transportation in the United States, there is no city with a population of more than 250 000 which is not on a navigable waterway, and such States as lack the necessary water routes show indeed a very low stage of development when compared with those which are more favorably situated. The writer does not know of any city, in the entire history of the world, which has had more than 300 000 inhabitants, which was not situated on a navigable water route. Railroads alone have never been able to build a large city.

Mr. Lavis also states that it is necessary for water transportation routes that business shall come to them, but why should not the railroads feed the water routes? Why should it not be said that water routes can reach the interior (through rail service) as it is now admitted that railroads reach the other side of the oceans (through water routes).

The statement that some cargoes, notably coal and grain, deteriorate from wet and dampness when moved by water, and that there are other inherent difficulties in connection with inland water transportation—such as increased hazard and insurance rate—is contradicted by the statement of the Chamber of Commerce of Rotterdam that during 1913 more than 4 000 000 tons of grain and more

than 6 000 000 tons of coal were moved on the Rhine to and from Germany. Mr. Bernhard.

In connection with the experience of Mr. Lavis in Argentina, which is very hard to consider properly without all the facts in connection therewith, the writer wishes to mention Alberta and Saskatchewan, in Canada, where the rivers are developing the country and supporting the railroads. Only recently, in a report on the Peace River, running from British Columbia through Alberta to Great Slave Lake, and on the Athabaska, the writer claimed that these rivers alone could develop the country, and that these water routes would never be fully supplanted by railroads.

Because the Santa Fé in Argentina was accepted by Mr. Lavis as not able to solve the transportation problem of that country, it is just as little proved that all the waterways in the world are out of place, as the writer's opinion that the Peace River in Alberta will always remain a serious competitor of the railroads proves that all water routes of the world will supplant railroads.

The great trouble with water transportation in the United States is its absence. Unknown makes unbeloved. There are very few people in the United States familiar with the problems of water transportation, and its almost total absence makes the number of its competent champions small. In lieu of these, it has an army of politicians who fight for "water improvement" for the sake of the spoils; a large number of well-meaning, poorly-led, and frequently ill-advised citizens, and a considerable legion of highly-trained and experienced transporters who can only regard water transportation from their standpoint in life—that of railroad men.

Inland water transportation is sadly needed, and the writer believes that it can be revived through the following measures:

First.—That waterways, harbors, bridges, and quarantine should be combined in a new department, with a cabinet officer as head; this department should employ engineers specially educated for this at West Point. These engineers, though commissioned officers in the United States Army, should not be under direct charge of the Army officials during the time they are engaged in this Waterways Department.

Second.—That the decision, which of the new waterways, if any, are a commercial necessity, should not be made by such engineers, but be considered by a group of transportation experts specially employed by the Government for such purpose.

Third.—That a definite, concise, uniform plan—a system of waterways improvement and development, etc.—should be prepared by this Department for the entire United States, Congress to modify or improve this and appropriate a yearly lump sum to be spent in approaching this ideal of approved waterways.

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Fourth.—That all States and communities are in duty bound to build inland water terminals, just as they now feel obligated to build sea terminals (harbors).

Fifth.—That an educational campaign, in municipalities, for the building of belt railroads, warehouses, and other facilities of communication to the harbor, should be instituted.

Sixth.—That this measure shall provide for the public ownership of the river front. This, however, is not to be accepted as being antagonistic to private ownership, or at least to private leases ranging from periods not shorter than 20 years, unless so desired by the tenant.

Seventh.—That it is an absolute necessity that the abuse of the rail rate to meet water competition, shall be stopped; that successful water transportation means rail rates tied down to fixed charges per ton-mile, plus terminal charges, and enforced exchange of freight between water and rail terminals.

Eighth.—That this measure is opposed to the licensing of pilots. or to the compulsory employment of licensed officers.

Ninth.—That this measure provides for a uniform and Federal quarantine law.

Tenth.—That our waters and coasts should be open for navigation to any nationality or trade. It is necessary that the United States shall make use of all its waterways; this will bring work to our shipyards and supply houses, and employ, under the present laws, American labor. The stimulation of industries depending on water transportation brings life to it.

Mr. Low.

EMILE LOW,* M. AM. SOC. C. E. (by letter).—It may be of interest, in connection with this paper, to give the cost of transportation on the present Erie Canal, owned by the State of New York, and, for this purpose, a fleet of canal-boats, consisting of one steamer and five consorts, is used. The information was obtained from Mr. James Shean, of New York City, who owns and operates such a fleet, composed of the Steamer *J. W. Morse*, and five consorts.

The size of the canal-boats on the Erie Canal, is governed absolutely by the size of the locks, the boat completely filling them. The boats are 98 ft. long, 18 ft. beam, and have 6 ft. draft.

The capacity of each of the consorts, so-called, on this draft, is 8 000 bushels of wheat, and, for the steamer, 4 500 bushels, so that the fleet mentioned carries 44 500 bushels of wheat (1 335 tons), which, at $4\frac{1}{2}$ cents per bushel for carriage, Buffalo to New York, nets $2 002.50.

The usual time from Buffalo to New York, 500 miles, for a steamer fleet, is 9 days, the return trip occupying about the same time. Usually, the round trip takes 1 month, allowing for loading and unloading at

* Buffalo, N.Y.

the termini. Under ideal conditions seven round trips can be made during the navigation season, May-November, inclusive. Mr. Low.

The arrangement of a 6-boat fleet in the canal, Buffalo to the Hudson River, at Waterford, is as follows: The steamer pushes one consort, and tows two pairs of consorts, 500-ft. lines being used, each pair of boats being closely coupled.

A full crew consists of eleven men, five of whom are discharged on arrival at Waterford, where the fleet is made up into a single tow, all boats being lashed together, the steamer towing all of them to New York on the Hudson River.

The approximate operating expenses for 1 month are as follows:

Steamer and consort:				
1 Captain or owner..........	1 month	=	$100.00	
1 Engineer.................	1 "	=	70.00	
1 Assistant engineer.........	1 "	=	35.00	
1 Wheelsman	1 "	=	75.00	
1 "	$\frac{2}{3}$ "	=	50.00	
				$330.00
First pair of consorts:				
1 Captain	1 month	=	$70.00	
2 Wheelsmen (each $50).....	$\frac{2}{3}$ "	=	100.00	
				170.00
Second pair of consorts:				
1 Captain	1 month	=	$70.00	
2 Wheelsmen (each $50).....	$\frac{2}{3}$ "	=	100.00	
				170.00
Subsistence:				
6 men.............	1 month at $18.00	=		108.00
5 "	$\frac{2}{3}$ " " 12.00	=		60.00
50 tons, hard coal..........	" 3.15	=		157.50
Oil and waste..................		=		4.50
Total cost per month....................				$1 000.00

In the case cited, the owner's wife acts as cook for the crew of the steamer and first consort. It will be noted that no allowance is made for her wages. Subsistence is calculated at 20 cents per meal, or 60 cents per day, being $18 per month, for members of the full-time, and $12 for the part-time crew.

The sums paid to the captains of the first and second pair of consorts include the wages of the cooks, usually the wives of the captains, who are paid 20 cents for each meal furnished the crew, so that the amount previously calculated, $168, closely approximates the cost of subsistence.

Mr. Low. In addition to the expense given, there are the following additional charges:

1.—Trimming cargo at Buffalo, $1.00 per 1 000 bushels, for cargo of 44 500 bushels of wheat = $44.50.

2.—Trimming cargo to elevator leg at New York, $1.50 per 1 000 bushels, for cargo stated = $66.75.

3.—Marine insurance, in accordance with classification of canal-boats:

On first-class canal-boats = $0.40 per $100 value.
" second-class " = 0.60 " " "
" third-class " = 0.90 " " "

The insurance is for the gross value of the cargo plus the freight.

With wheat at $1.00 per bushel, the gross value of the cargo would be $44 500, freight at 4½ cents per bushel would be $2 002.50, a total of $46 502.50. As many of the canal-boats are listed as second-class, the insurance, at 60 cents per $100 value, would amount to $279.

In addition there is a commission or brokerage charge of 5% on the freight value, in this case, $100.10.

Recapitulating we have the following statement:

Operating cost	$1 000.00
Trimming cargo at Buffalo	44.50
" " " New York	66.75
Insurance on cargo and freight	279.00
Commission and brokerage	100.10
Incidentals	109.65
Total	$1 600.00

At 4½ cents per bushel of wheat, the rate per ton is $1.50. On east-bound trips the fleet carries a full or capacity cargo. Returning west, 1 000 tons is the usual cargo. During the last two years, much flaxseed has been carried west, for which the rate was 85 cents per ton, the shipper paying all charges, thus netting the carrier $850.

We then have the total receipts, as follows:

East-bound freight	$2 002.50
West-bound "	850.00
Total	$2 852.50

Adding the flaxseed (35 000 ± bushels) to the wheat, we have a total quantity both ways of 80 000 bushels of grain, costing the carrier exactly 2 cents per bushel to transport, and giving him a profit of $1 252.50 per round trip, or $208.75 per boat.

The cost of a canal steamer is about $13 000, and that of an ordinary canal-boat, or so-called consort, $3 400, a 6-boat fleet costing $30 000. Mr. Low.

The annual expense of operation and the receipts are as follows:

6% interest on investment			$1 800.00
2% insurance			600.00
3% ordinary repairs			900.00
6 months operating cost at	$1 600 =	$9 600	
1 " " " "	1 000 =	1 000	10 600.00
5 " (winter) " " "	200 =		1 000.00
Annual outlay	=		13 900.00
6 months freight at $2 852.50	=		17 115.00
Difference	=		$3 215.00

Allowance is not made for extraordinary repairs, for bad or lean years, or for a lesser freight rate than that given, that is, $4\frac{1}{2}$ cents per bushel of wheat.

After paying 6% interest on the investment, the remainder, invested in a sinking fund, will provide a sufficient amount to purchase a new fleet, after an expiration of 20 years, which is about its life. Insurance rates increase with the deterioration of the boats, which also renders them unfit for carrying grain, and then other commodities, such as lumber, pig iron, stone, gravel, etc., must be sought.

The following is an attempt to give the cost of "horse" boats, for one fleet of three boats:

1 Captain		$100.00	
1 Wheelsman	1 month	75.00	
1 "	$\frac{2}{3}$ "	50.00	
1 Extra wheelsman	$\frac{2}{3}$ "	50.00	
2 Drivers	$\frac{2}{3}$ " at $50	100.00	
			$375.00
Subsistence:			
2 men	1 month at $18	$36.00	
4 "	$\frac{2}{3}$ " " 12	48.00	
			84.00
Keep of 6 horses or mules, $1.00 per day each, 30 days		$180.00	180.00
Towing, Waterford to New York		$50.00	
" Buffalo		25.00	
			75.00
Total			$714.00

Mr. Low.

Cargo, 24 000 bushels of wheat:

Trimming at Buffalo........at $1.00	$24.00		
" " New York.... " 1.50	36.00		
Insurance on cargo and freight..........	150.48		
Commission and brokerage..............	54.00		
Incidentals	21.52		
		286.00	
Total		$1 000.00	

Freight, east-bound: 24 000 bushels wheat at $4\frac{1}{2}$ cents	$1 080.00
Freight, west-bound: 500 tons at 85 cents........	425.00
Total	$1 505.00

Total freight carried, 42 000 bushels, costing the carrier 2.4 cents per bushel, and giving him a profit of $505 per round trip, or $168.33 per boat.

Cost of fleet:

3 Canal-boats, at $3 400..........	$10 200
6 Horses or mules, " 200..........	1 200
	$11 400

Annual expense:

6% on investment..................	$684.00
2% insurance	228.00
3% ordinary repairs.................	342.00
7 months operating cost at $1 000..........	7 000.00
5 " (winter) horses..............	300.00
" " boats..................	200.00
Total	$8 754.00

Receipts:

6 months at $1 505..................	$9 030.00
Difference	$276.00

Mr. Harts.

WILLIAM W. HARTS,* M. AM. SOC. C. E. (by letter).—It seems to be a rather familiar criticism, not altogether unfounded, it must be admitted, which has been advanced by students of the development of our rivers as common carriers, that we do not accomplish as much with our money here as some foreign countries where the systems of selection of waterways for improvement and the amounts expended

* Washington, D. C.

are the subjects of what is supposed to be much more careful scrutiny than here. It was the writer's purpose to point out how some of the economic features of our systems might be eliminated and the profitable features developed further, all with a view to obviating in the future the justice of some of these criticisms. Mr. Harts.

It should be kept in mind that in most foreign countries the choice of what rivers to propose for traffic purposes is by Nature already made, in most cases there being but few from which to select. Furthermore, the conditions surrounding their use there are so different from what they are here that the inestimable service that many of our rivers have already rendered in the development of a new land is likely to be overlooked when their present idle condition is in mind. Most foreign countries charge a tonnage tax on all water freight. Many of them allot a large part of the first cost of construction to localities benefited. Whether or not these requirements are wise, in the long run, the river navigation of this country is free of both these burdens. Then the density of population is much greater there than here; the rivers were used for commerce, and the people grew to be accustomed to them long before railways were the efficient carriers they have become of recent years; and furthermore, the railway systems there are in the main so subject to Government control that they are often compelled by order to fix their rates so that the rivers will get some undue preference. These considerations put a somewhat different aspect on the economics of this whole subject.

Few persons who are well informed on river improvement will begrudge the greater part of the expenditures made for interior rivers in the past when they consider the great effect on the expansion and growth of this immense country, but what should seem wise to them now would be to scrutinize anew our waterways and select for our present expenditures only those the future usefulness of which is assured. For the future, it seems plain that we should restrict expenditures on those like the Kentucky, the Missouri, and the Mississippi, on which the conditions of channels for navigation are already far in advance of the necessities of present commerce. In this way we may even yet remedy some of the present defects and increase our average of efficiency.

Mr. Lavis seems to feel that our system of channel building is haphazard. Perhaps this idea may arise from some misconception of the way many of our streams were developed. The whole growth of our land might, in the same sense, be considered to have been haphazard, but under the visible surface the little understood but inevitable law of economics has been working incessantly and surely. The wish of the writer now is to apply the same law of economics to the future of our river systems and recognize more clearly the limits of the work required. The Board of Rivers and Harbors, through

Mr. Harts. whose hands every new project must pass for examination and recommendation, is already restricting the expenditures for certain streams, but the people as a whole must see the reason for this and must support it if this work is to be done efficiently, economically, and wisely. It is not very difficult to determine the worth of any river for navigation, although predicting its future is a much harder task; but, like any other engineering project, intelligent analysis will point the safe way.

Major Burgess is correct in his discussion. Railroads should not be permitted to discriminate against rivers by favoring one locality at the expense of others, for this, as he has so well pointed out, is contrary to good public policy. Happily this tendency is believed to be proving less important, and the control of rates by the Interstate Commerce Commission may, it is thought, be trusted ultimately to regulate even this unfairness. Already the milling in transit privilege is ended, and other restrictions will probably soon follow, if unfairness exists. It is too much to expect all these new changes to be made at once, but the steps already taken may be assumed to be a good index to the future, and the future is what we must keep in view.

There is everywhere a large class of critics, and they impatiently call on other people for the immediate advantages of a Utopian prospect where everything is as it should be. Whether these dreams would turn out to be as successful as the anticipation, only a trial could prove. It seems safe to say that it is usually the part of wisdom to point out the improvements that are practicable in the existing order of things, thus building with assurance of benefit on what already is well under way, rather than to pull down everything for a fresh start. Thus we may often avoid the dangers of the radical and hazardous uprootings of the eager but short-sighted reformer.

Mr. Bernhard thinks we have too many sizes of locks and channels. He thinks that the standard railroad track and car point out a lesson for our river engineers. This is probably an unconsidered reflection. Economy would certainly indicate the unwisdom of the construction of as large locks and as deep and wide channels on smaller tributaries of the Ohio, for example, as those in the main stream. Standardizing is desirable, up to a certain point, and the present varying sizes of locks and dimensions of channels are perhaps greater than desirable, but to give the impression that there should be a standard lock or standard channel and a uniform boat suitable for all purposes is manifestly following the railroad analogy too far. Mr. Bernhard also charges lack of terminals with being responsible for the diminishing inland navigation. Adequate terminals are necessary on all transportation lines, but they do not make traffic; they only accommodate it. If commerce were active enough on our interior rivers, terminal facilities would keep pace with its requirements. We may be sure

of this, from the inevitable economic law. The lack of suitable terminal facilities is only an indication of the lack of the necessity for them. On the Great Lakes, where a large commerce is found, terminal facilities are more advanced than in many of our sea ports. Mr Harts.

With regard to Mr. Bernhard's conclusions, many of them would doubtless be desirable, but others are open to objection. The creation of a Federal Waterways Bureau or department has been suggested many times before. Can any one believe that a new department with a Cabinet officer at its head would lessen those burdens which river transportation is now struggling under? Whenever there is again a sufficient profit in river traffic, we shall see new and suitable terminals, comprehensive plans for river development, improved steamboats and barges, and a rejuvenation of an old industry; but, until that day comes, we may discuss the case often and long, and ply the patient with all sorts of supposed remedies, but the trouble will be found to be unimproved until the economic conditions on our streams grow better.

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Paper No. 1347

THE DESIGN OF HYDRO-ELECTRIC POWER PLANTS

By J. D. Galloway, M. Am. Soc. C. E.*

With Discussion by Messrs. William P. Creager, E. Newman, H. Homberger, Arnold Pfau, and J. D. Galloway.

Synopsis.

The object of this paper is to examine certain of the problems relating to the design of hydro-electric power-plants, from the standpoint of the engineer in charge of the entire plant. The paper deals with hydraulic problems, and refers only incidentally to the electrical features. The detailed design of water-wheels is also omitted.

The particular features treated are: (1) the general conditions affecting the design of plants; (2) the conduit; (3) the penstock pipes; and (4) the station design.

In this examination, reference is made to the following points:

The load factor and its influence on design;

The relation of an auxiliary steam plant to the entire system, supplementing the work of the hydro-electric plant;

The rules of economy governing the design of all parts transmitting and transmuting energy;

The design of water conduits and regulating reservoirs, with data on the friction factors of flumes, and examples of conduit design;

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* Presented at the meeting of May 19th, 1915.

The design of penstock pipes, with tables of the weight of riveted pipes and the auxiliary parts, such as valves, etc.;

The number of units in a plant, considering small stations, large stations, and stations as parts of a system;

The limitation of water-wheels and generators;

The general relations of head, power, and speed, with derivation of formula for specific speed;

The limitations of impulse-wheels, their field of use, examples of installations, and the value of the specific speed;

The limitations of turbines, their field of use, examples of installations, and the value of the specific speed;

The selection of a water-wheel and generator for different conditions;

Various curves illustrating the limitations of water-wheels;

Some general remarks on station design and the arrangement of auxiliaries.

The conclusions of the paper are as follows:

The load factor has a very important bearing on the design. A reserve steam station is necessary at the receiving end of a long line, and should be operated over the peak, in order to raise the load factor, at the hydro-electric station, up to an amount of 60 or 75 per cent. This will reduce the installation cost of the hydro-electric plant and transmission line in an amount sufficient to build the steam plant.

The rules of economy give results which can be only guides to an intelligent design of the energy conduits. Hence, judgment must largely govern the slope of water conduits, the size of penstocks, and the weight of copper in the line.

A regulating reservoir is necessary, in high- and medium-head plants, at the end of any free-flowing conduit. The surge chamber on a closed conduit is to be resorted to only in the absence of a regulating reservoir.

The number of power units in a small station should be at least three, and in any station not more than four or five, unless the total available power is greater than can be generated by five units of maximum size possible for the particular case.

Impulse-wheels should preferably be limited to one nozzle on one wheel and to a value of 15 of the ratio of pitch diameter of wheel to

diameter of jet, giving a specific speed of 3.48, maximum, under normal working conditions.

Turbines have a present maximum limit of specific speed of 82.3 (366 metric) and a minimum limit of 9.4 (42 metric), but future designs may enlarge these limits.

Impulse-wheels are the only available motors for heads of more than 1 000 ft. (305 m.). Turbines, except for very small wheels, are the only motors for low heads, 200 ft. (61 m.). Between these limits, both wheels are available, with the tendency to use turbines on higher and higher heads as the size of the unit increases. This tendency is restricted by limitations of generator speed.

The limit of power developed by one unit has not been reached, and larger units may be expected in the future. The largest single hydro-electric generator is the 17 500-k.v.a. unit at Big Creek, Cal.

INTRODUCTORY.

This paper is a review of some of the problems which arise in the design of hydro-electric power plants. It is limited to a discussion of the hydraulic features, and no reference is made to the electric parts, except in so far as they affect the design of the hydraulic elements. In the same manner, there is no discussion of the detailed design of water-wheels. The design of separate parts of a plant, such as generators, water-wheels, the pipe line, etc., has been treated in the publications of the various societies.

The object of this paper is to present the problem of design from the standpoint of the engineer who has to consider the entire plant. Due to the nature of the work and to the segregation of engineering into three separate branches, Civil, Mechanical, and Electrical, all of which have to do with hydro-electric plants, there has been considerable discussion of the separate parts of such plants in the publications of the various societies, very often with little consideration of the relation of the machine to other parts of the plant.

The engineer whose province is to assemble the entire plant into a working whole is necessarily without the detailed knowledge possessed by the engineer who designs the turbine or the generator, and must rely on the latter for such information. On the other hand, it often happens that the engineer of one of the special parts is not in com-

munication with the engineer who builds another part, which in the completed plant is directly related thereto. The turbine designer may set forth requirements, say of speed, necessary from his standpoint, which the generator builder cannot meet. It is hoped that this paper may bring out discussion of some of the questions involved, and by each of the three divisions of those interested in hydro-electric plant design.

It is not deemed necessary to refer to the development of water supply and the storage of water, for though such problems are an integral part of power plants, their discussion would extend the paper beyond proper limits and divert attention from the points on which it is desired to dwell.

Hydro-electric developments cover ranges in head of water from 10 up to 3 000 ft. or more. Although there are many differences of detail, the essential features are the same in all cases. Some of the problems of high-head plants are absent from those of low heads, and this is the excuse for dwelling more at length on the subject of high-head design. The particular features treated herein are: (1) the general conditions affecting the design of plants; (2) the conduit; (3) the penstock pipes; and (4) the station design.

General Conditions Affecting Plant Design.

The Load Factor.—Before taking up the subject of the design of parts, consideration must be given to the load factor. By this term is meant the ratio between the average power demand and the maximum or peak demand during any period in which a cycle of events is gone through, generally a day. A lighting load factor may be from 15 to 25%, that of a street-car system 50%, and that of a mill or a mine, running night and day, 80 to 90 per cent. A large power system has a factor which is the composite of all its separate load factors. Such factors vary considerably, as shown by Table 1.

TABLE 1.—Load Factors.

Niagara Falls Power Company	81.0%
Commonwealth Edison Company, Chicago	40.0%
New York Edison Company	34.7%
Philadelphia Electric Company	34.4%
Boston Edison Company	32.5%
Pacific Gas and Electric Company, California	59.0%
Great Western Power Company, California	70.0%
Sierra and San Francisco Power Company, California	50.0%
Pacific Light and Power Company, California	47.8%
Los Angeles Gas and Electric Company, California	40.0%

In order to emphasize some points of design by examples, a load factor of 50% will be chosen for this paper. A load curve of a street-car system producing this load factor will also be used.

The load factor affects all parts of the design, and is the governing condition in many cases. The water at an elevation contains a certain amount of energy, static when in the reservoir. This energy is carried by the water at first through canals and flumes, or by the natural bed of the stream, is transmuted into electric energy at the power-station, and in this form is transmitted to a distance by the lines. In all these various conduits containing and conducting the energy, the size and resulting cost is a direct function of the maximum amount of energy transmitted at any one time. Hence it is that the maximum demand fixes the size and cost of the conductors. As the average flow of energy is a measure of the resulting revenue, the load factor is of vital importance in the design, as it may be shown that a projected development may be unprofitable if the load factor is too low.

The possibility of using a reserve steam station to carry the peak of the power load should also be considered. On Fig. 1 is shown a typical load curve of a street-car system, the changes in the curve being taken by hours. The load factor of the curve is 50% and the power developed at any one time is shown as a percentage of the peak load. If the peak load is carried by the hydro-electric station, the entire equipment must be designed with reference to the maximum output. If, on the other hand, a reserve steam station is installed for emergencies, then use can be made of this station to reduce the peak on the hydro-electric station, or, in other words, to raise its load factor.

By allowing the steam station to take 40% of the system peak, the hydro-electric station must carry 60% of the peak. In this case the steam station generates some of the energy, and if a given quantity of water is available for the hydro-electric station, the total energy delivered by the two plants is increased over that of the hydro-electric station by the small amount generated at the steam station during peak load. On the curve in question, the hydro-electric station, by carrying 60% of the system peak, gains a load factor of 75% and at the same time generates 90.6% of the total

energy. The steam station carries 40% of the peak, has a daily load factor of 11.75%, and generates 9.4% of the total energy.

In such a system as this, if the hydro-electric plant has an average capacity of 15 000 kw., the peak load on a load factor of 75% is 20 000 kw.; but if the load factor is reduced to 50%, as given for the entire system, the peak load becomes 30 000 kw. This would require, in the latter case, an addition of 10 000 kw. of installation more than that necessary for a load factor of 75 per cent. As this

LOAD CURVE ILLUSTRATING LOAD FACTORS AND INFLUENCE OF THE OPERATION OF AN AUXILIARY STEAM PLANT.

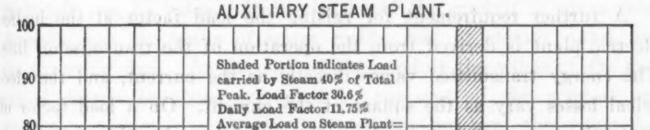


FIG. 1.

addition of installation affects the design and size of pipe lines, water-wheels, generators, transformers, and transmission lines, a large difference in cost is represented by the difference of 10 000 kw. At $100 per kw., this would amount to a difference of $1 000 000, which would build, under modern conditions, a steam plant of 20 000 kw., or as large as would be necessary in such an installation, as an emergency plant. The additional cost of operation, over the standby charges, would be slight, and is represented practically by the fuel.

For this there is compensation in the sale of the steam-generated energy.

These statements would be varied by every different kind of load curve, but in general they remain true. Depending on the form of the curve, and with present fuel prices, it can be shown that when a hydro-electric plant, with an ordinary length of transmission line, requires (for continued operation) to be supplemented by a steam plant, it is the best practice to operate the steam plant over the peak and raise the load factor on the hydro-electric plant up to the neighborhood of from 60 to 75 per cent.

A further requirement for raising the load factor at the hydro-electric plant is derived from the operation of the transmission line. The energy transmitted varies directly as the current, and the electrical losses vary as the square of the current. On a load factor of 50%, the energy transmitted at peak load is twice that at average load, at which time the losses would be approximately four times that of average load. On a 75% load factor, the losses at peak would be less than twice those at the average.

This entire subject becomes one where the conditions surrounding each particular case must govern. It is mentioned here as a major point affecting the design of hydro-electric plants.

The Rules of Economy.—The first rule of economy is that, in any given installation, the design should be governed by the consideration that the amount of the investment should be the least possible within the limits of good construction, other things being equal. In some cases where there is an abundance of water, it is not necessary to strive for the highest efficiency of the various parts, as the losses are not vital. This statement is true during a large part of the year in most plants, but, as a general rule, the power developed is measured by the minimum flow of water, and hence the economy of design holds true.

The second rule of economical design can be stated as follows: Any conduit carrying energy satisfies the requirement of minimum cost when the sum of the annual cost of conduit plus the value of the energy annually lost is a minimum. This rule governs all the conduits of the energy: ditches, flumes, tunnels, pipes, and transmission lines. As will be shown later, considerable latitude is possible in any

design, without departing far from the rule, and here the governing factor is to reduce the cost of the investment to a minimum.

The Value of Power.—In applying the rule of economy, the value is not that for which it is sold. It is the cost alone that measures the value as far as design of parts is concerned. If the design is made on this basis, it will be properly proportioned to the actual sale value of the energy.

The value of the energy at any place is a variable, depending on the position it then occupies in the course of its transmission from the source to the place of use. Judged by cost, the energy in water as it falls has no value. At the intake of the conduit it has a value depending on the value of water rights, cost of storage dams, and the diverting dam. At the end of the conduit the value of the contained energy has been increased by the cost of the conduit and regulating reservoir. At the high-tension bus-bars the value has been further increased by the cost of penstocks and power-station equipment, and, at the delivery end, there has been added the cost of the transmission line and the sub-station. A corollary of this proposition is: if money is to be spent to save energy by reducing losses, it should be apportioned, not along the course, but near the receiving end. If a given sum of money will save a given amount of energy otherwise lost, it is much better to apply it at the receiving end. If applied near the origin, say on the conduit, the energy saved must be transmitted through all the other parts of the system along the entire course. It must also be borne in mind that the amount of energy conveyed grows constantly less by losses, and is decreased to about 60% of the original amount at the receiving end.

In practice it is impossible to know the cost before the design is prepared, and assumptions must be made. If the cost of an installation is such that the resulting cost of energy amounts to 0.5 cent per kw-hour, the values may be apportioned at 0.2 cent for the conduit calculations, 0.3 cent for the penstocks, and 0.4 cent for the transmission line. Such values will give results of sufficient accuracy.

The Conduits and the Regulating Reservoir.

In many low-head plants, where the quantity of water used to develop a given amount of energy is large, the installation is limited in maximum output by the natural flow of the stream, supplemented

by such pondage as may be obtained by flash-boards or by the dam itself. In plants with high heads, where the water is brought a considerable distance in a conduit, the quantity of water is much less for a given development of power, and hence it is possible to control the flow delivered to the wheels by a regulating reservoir. If such a reservoir is available, the varying demand for water can be supplied from it, and the conduit can be designed for the average flow of water. In fact, in any open conduit of considerable length, it is hardly possible to regulate the flow at the intake to correspond to the varying flow at the power-station. In the case of closed conduits, such as pipes or pressure tunnels, surge chambers of proper design will obviate the difficulties of operation with a long penstock, but will not eliminate the necessity of designing the conduit for the maximum demand, thus largely increasing the cost. The regulating reservoir has the merit of reducing the cost of the conduit, of performing all the functions of a surge chamber, and, if large enough, furnishes an emergency supply of water when the conduit may be out of service. In these respects, it is far superior to a surge chamber, which should only be used when nothing else is feasible.

The Conduit Design.—The conduit may be either open or closed. The open conduit may be built of flumes, open canals, pipes crossing canyons, or tunnels. The formulas of Kutter or of Bazin may be used in determining the section, but that of Kutter is in most general use in America. For canals in earth, the coefficients given by Kutter may be used. If canals are lined with concrete, loss of water is prevented and the section is reduced by decreasing the friction. A value of $n = 0.014$ is believed to represent the concrete-lined canal or tunnel after it has been in use, although the value is lower in new, clean canals. For flumes built of surfaced lumber, with battens covering the cracks, a value of $n = 0.013$ to 0.014 is conservative. As there are few published data on large flumes, there are given in Table 2 the results of gaugings of two flumes made in 1906 by Charles D. Marx, President, Am. Soc. C. E., Mr. L. M. Hoskins, Wynn Meredith, M. Am. Soc. C. E., and the writer, preparatory to the design of the 14-mile flume of the Sierra and San Francisco Power Company. The Floriston flume had been in use about 10 years. It was 7 ft. 6 in. deep and 10 ft. 3 in. wide, built with surfaced lumber and rough $\frac{5}{8}$ by 4-in. battens over the cracks. The

Fleish flume was nearly new, 6 ft. 0 in. deep, 10 ft. 0 in. wide, built with surfaced lumber, and the cracks were closed by a tongue, so that there were no obstructions in the flume.

TABLE 2.—GAUGINGS OF FLUMES.

	Floriston Flume.			Fleish Flume.
	Full flume.	Full flume.	Half flume.	Full flume.
1906	July 7th	July 9th	July 9th	July 8th
Length	600 ft.	300 ft.	300 ft.	500 ft.
Width	10.23 ft.	10.23 ft.	10.23 ft.	10.07 ft.
Depth	7.58 ft.	7.05 ft.	5.46 ft.	6.08 ft.
Number of battens	16	16	14	None.
Net area	77.21 ft.	71.79 ft.	55.57 ft.	61.23 ft.
Wetted perimeter	27.39 ft.	26.33 ft.	22.90 ft.	22.23 ft.
Hydraulic radius	2.819 ft.	2.727 ft.	2.427 ft.	2.754 ft.
Slope	0.00062	0.00064	0.000397	0.000426
Mean velocity	5.45	5.62	4.37	5.12
Q, in cubic feet per second	420.8	403.5	242.8	313.5
$c = \frac{v}{\sqrt{rs}}$	130.4	134.5	140.8	149.5
n in Kutter's formula	0.0135	0.0130	0.0125	0.0117

The shape of the cross-sections of the canals will depend on the material in which they are excavated, and also on the slope of the hillside. For economy in excavation, canals in level ground, or ground with slight slope, should be relatively wide and shallow, but on ground of steep slope, they should be relatively deep and narrow. On steep slopes there should be a berm on the upper side in order to prevent slides of material into the canal, which, especially if the canals are lined, reduces the carrying capacity materially.

Economical Slope for Conduit.—To illustrate the method of calculation of an economical slope for a conduit, one was assumed which would carry 350 sec-ft. of water, and be lined with concrete. To facilitate calculation and to produce uniform results, the section of the canal was given dimensions which are functions of the depth; with these, the slopes necessary to carry the water, with different sections, were determined. The cost of the canal, in lengths of 1 000 ft. of different sections, was then estimated. The annual value of the power lost by 350 sec-ft. of water falling 1 ft., or 29.7 kw., was taken for four assumed unit values: 0.2, 0.3, 0.4, and 0.5 cents per kw-hour, giving total values of \$520, \$780, \$1 040, and \$1 300, respectively, for the power per kilowatt-year. Table 3 is made up of these data, interest on the cost being taken at 6 per cent.

TABLE 3.—SUMMATION OF ANNUAL COST OF 1 000 FT. OF CANAL AT 6% ANNUAL INTEREST, AND VALUE OF ENERGY LOST IN DIFFERENT SLOPES AT $17.50, $26.25, $35.00, AND $43.80, PER KILOWATT-YEAR. 350 SEC-FT. OF WATER.

Depth of canal.	Cost, 1 000 ft. of canal.	6% annual interest.	Fall in 1 000 ft.	Val. $17.50 Kilowatt-Year.		Val. $26.25 Kilowatt-Year.		Val. $35.00 Kilowatt-Year.		Val. $43.80 Kilowatt-Year.	
				Value, power lost.	Power plus interest.	Value, power lost.	Power plus interest.	Value, power lost,	Power plus interest.	Value, power lost.	Power plus interest.
4.8	$6 960	$418	1.53	$796	$1 214	$1 195	$1 613	$1 595	$2 013	$1 985	$2 403
5.0	7 590	456	1.19	619	1 075	929	1 385	1 238	1 694	1 545	2 001
5.2	8 060	484	0.99	515	999	772	1 256	1 030	1 514	1 285	1 769
5.4	8 590	515	0.79	411	926	616	1 131	825	1 340	1 025	1 540
5.6	9 000	540	0.66	343	883	515	1 055	687	1 227	858	1 398
5.8	9 530	572	0.55	286	858	429	1 001	572	1 144	715	1 287
6.0	10 050	603	0.46	239	842	359	962	479	1 082	598	1 191
6.2	10 450	627	0.39	203	830	304	931	405	1 032	508	1 135
6.4	10 820	650	0.33	171	821	257	907	343	1 003	429	1 079
6.6	11 480	689	0.27	140	829	210	899	281	970	351	1 040
6.8	12 120	728	0.23	120	848	178	906	239	967	300	1 028
7.0	12 470	748	0.20	106	854	160	908	213	961	267	1 015
7.2	12 950	777	0.18	93	870	140	917	187	964	234	1 011
7.4	13 500	811	0.15	78	889	117	928	156	967	195	1 006
7.6	13 780	826	0.14	73	899	109	935	145	971	182	1 008
7.8	14 170	850	0.13	67	917	101	951	135	985	169	1 019
8.0	14 920	896	0.11	57	953	86	982	115	1 011	143	1 039

The data in Table 3 are set forth graphically on Fig. 2. A corresponding set of calculations for a tunnel is shown on Fig. 3, where the section of the tunnel is given.

The minimum values of the sum of annual cost of conduit, plus annual value of power lost, are at the lowest points on the curves. It will be noted that the curves are very flat near the minimum value. This will allow of considerable variation from the determined economical slope, and this variation should be toward the steeper slope, with smaller cross-section and lesser cost, in order to reduce the investment.

Flumes.—As a rule, it is not possible to vary the section of a flume to suit the slight variations of slope. If the flume is in short lengths between divisions of the canal, it can be placed on the same grade. If the conduit is a long flume, a few calculated sections will determine the economical slope. Fig. 4 shows a form of flume common in western America. It is built with "boxes" 16 ft. long.

General Statements.—The foregoing rule of economy cannot always be applied. A favorable site for a diverting dam and the location

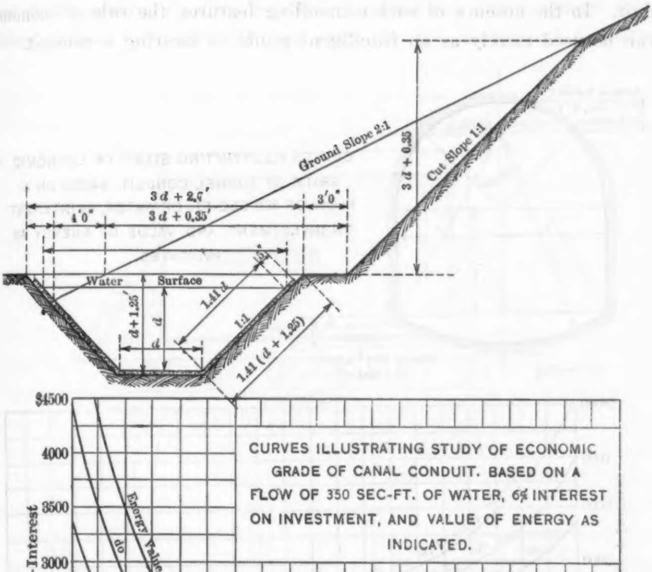

Ground Slope 2:1
Cut Slope 1:1
3 d + 0.35
3 d + 2.0'
3'0"
4'0"
3 d + 0.35'
Water
Surface
d+1.25
d
1.41 d
1:1
1.41 (d + 1.25)
CURVES ILLUSTRATING STUDY OF ECONOMIC GRADE OF CANAL CONDUIT. BASED ON A FLOW OF 350 SEC-FT. OF WATER, 6% INTEREST ON INVESTMENT, AND VALUE OF ENERGY AS INDICATED.
Energy Value $45.80 per Kilowatt - Year.
$35.00
$26.20
$17.60
do
Annual Value of Energy + Interest
$4500
4000
3500
3000
2500
2000
1500
1000
500
$0
.00398
.00304
.00241
.00188
.00153
.00119
.00099
.00079
.00066
.00055
.00046
.00039
.00033
.00027
.00023
.00020
.00018
.00015
.00014
.00013
.00011
Slope of Canal
4.0
4.2
4.4
4.6
4.8
5.0
5.2
5.4
5.6
5.8
6.0
6.2
6.4
6.6
6.8
7.0
7.2
7.4
7.6
7.8
8.0
Depth of Canal (See Figure).

FIG. 2.

of a possible regulating reservoir often determine the grade of the conduit. In the absence of such controlling features, the rule of economy can be used merely as an intelligent guide in locating a conduit.

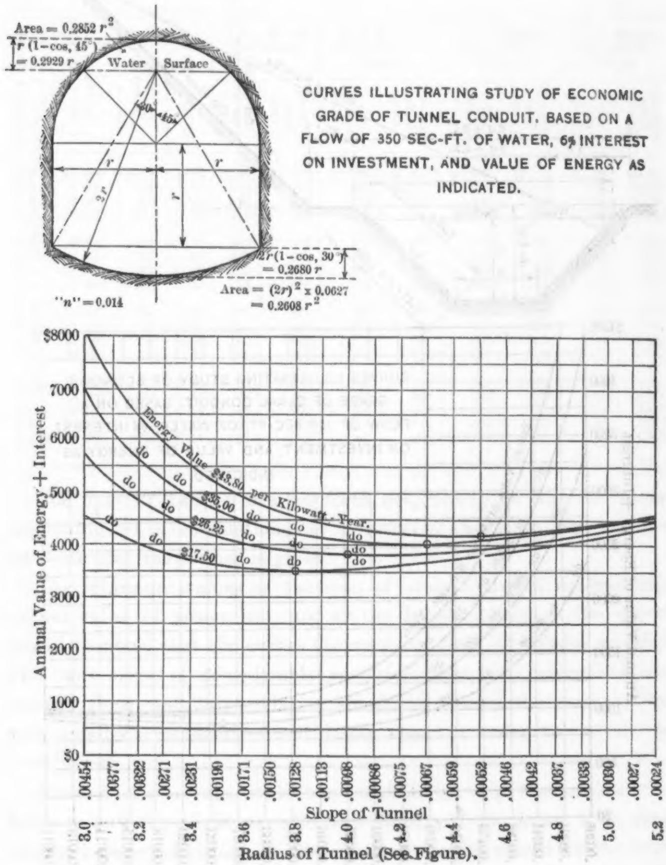
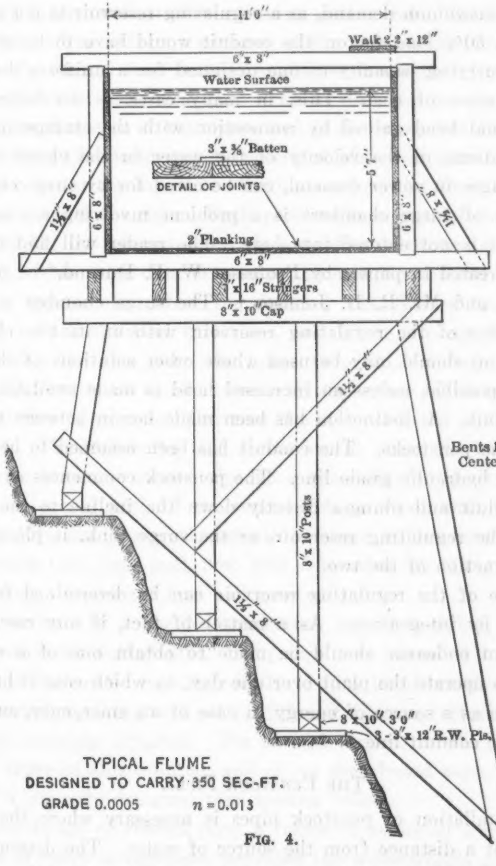


Fig. 3.

In the case of a closed conduit, consisting of a pipe or pressure tunnel, the economical section and slope would be determined in the same manner. The late Arthur L. Adams, M. Am. Soc. C. E., deduced the rule that a pipe satisfies the rule of economy when the

value of the energy lost in friction equals four-tenths of the annual cost of the pipe line.* This is merely a special case of the general rule. Other writers have also deduced rules for the economical diameter



TYPICAL FLUME
DESIGNED TO CARRY 350 SEC-FT.
GRADE 0.0005 $n = 0.013$

FIG. 4.

of pipes. In the application of these rules, the diameter of the pipe must be assumed, and, as the weight and resultant costs of pipe do not vary at a uniform rate with the diameter, it is believed that a

* *Transactions*, Am. Soc. C. E., Vol. LIX, 1907.

better way is to calculate a series of pipes in the manner illustrated herein, and determine which is the most economical in that way.

This disadvantage of the closed conduit, which is laid near the hydraulic grade line, lies in the necessity of designing the sections to suit the maximum demand, as a regulating reservoir is not possible. Thus, on a 50% load factor, the conduit would have to be of nearly twice the carrying capacity of one designed for a uniform flow, with resulting excess of cost. This, in many cases, is overbalanced by the additional head gained by connection with the storage reservoir. The fluctuations in the velocity of the water in the closed conduit, due to changes in power demand, can be cared for by surge chambers. The design of surge chambers is a problem involving a number of factors, and is not entered into here. The reader will find the subject fully treated in papers by Professor W. F. Durand,* of Stanford University, and Mr. R. D. Johnson.† The surge chamber is merely a special case of the regulating reservoir, without its two chief advantages, and should only be used where other solutions of the problem are impossible, unless an increased head is made available by the closed conduit. A distinction has been made herein between the conduit and the penstocks. The conduit has been assumed to be laid at or near the hydraulic grade line. The penstock commences at the end of the conduit and plunges directly down the incline to the power-station. The regulating reservoir, or the surge tank, is placed at or near the junction of the two.

The size of the regulating reservoir can be determined from the load curve by integration. As a matter of fact, if any reservoir is provided, an endeavor should be made to obtain one of a capacity sufficient to operate the plant over one day, in which case it has additional value as a source of energy in case of an emergency, such as a break in the conduit line.

The Penstock Pipes.

The installation of penstock pipes is necessary where the power-station is at a distance from the source of water. The determination of the economical size of the pipe can be made by selecting a number of possible diameters and calculating the best by the rule of economy.

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* *Transactions,* Am. Soc. Mech. Engrs., Paper No. 1353, 1912.

† *Transactions,* Am. Soc. Mech. Engrs., Vol. 30, 1908; and *Transactions,* Am. Soc. C. E., Vol. LXXVIII, p. 760.

However, a number of factors must be considered in selecting the size of the pipe. The formula of Mr. Adams, for economical diameters, makes the tacit assumption that the diameter is constant throughout the length, which assumption is usually incorrect. If a given loss is assumed at the beginning of the calculations, it should be apportioned to different sections of the pipe on the profile of the ground. In such a case it will usually be found that limitations of manufacture will determine that a small diameter at the lower end and a large diameter at the upper end is the most desirable. Again, the limitation of a minimum thickness of plate will provide a pipe of large diameter at the top, thus allowing most of the total loss to be at the lower part of the pipe. Each problem is so special that it is advisable to lay out several pipes on a profile, calculate the costs and losses, and from these data determine the most economical pipe.

Other factors also affect the design. The flow of water in the penstock follows the load variations, and if the average flow is used to determine the size, then the losses at peak load may cause such a drop in pressure as to affect seriously the speed regulation. If the peak-load flow is used to determine the size, then the pipes may cost too much. The higher the load factor on the plant, the better will be the service from the pipes. Speaking generally, the peak-load losses should not exceed 10% of the total head, in which case, if the load factor is 50%, the average losses will be about 2½ per cent. With low load factors, the peak-load flow will determine the size; with high load factors, the average flow will govern.

The writer has a decided preference for riveted-steel pipes over the imported lap-welded pipe. As prices have prevailed in the West, the cost of lap-welded pipe is from 10 to 15% less than that of riveted pipe, the design taking account of different friction factors to produce the same carrying capacity. The several failures of the welded pipe are the cause of this opinion, and, as to the riveted pipe, there is no case of a properly designed pipe having failed.

In the design of riveted pipe, a factor of safety of four, based on strength of joint, is sufficient, provided the water-wheels are equipped with by-passes such that pressure rises of more than 25% are not possible. Medium steel is used, having an ultimate strength of from 60 000 to 65 000 lb. per sq. in. (4 218 to 4 570 kg. per sq. cm.). Double-riveted lap-joints of 70% efficiency, and triple-riveted butt-strap

joints of 80% efficiency, are possible. In the case of the 70% joint, the unit stress in the plates would be 11 200 lb. per sq. in. (837 kg. per sq. cm.), giving a safety factor in the plate of about 5.5%; and, in the case of the 80% joint, the stress in the plate would be 12 800 lb. per sq. in. (900 kg. per sq. cm.), giving a safety factor of about 4.75% in the plate. For 30-in. (76-cm.) pipes or greater, the writer is accustomed to use lap-joints up to a thickness of plate of ½ in. (12.7 mm.), and butt-strap triple-riveted joints for greater thicknesses, and to use no plates less than ¼ in. in thickness. Figs. 5 and 6 give the static head which pipes of different diameters and thicknesses will stand. These are useful in preliminary layouts.

Table 4 gives the weight per foot of lap-riveted pipe, based on 6-ft. courses, the usual allowance for overweight of rolled plates, the calculated weights of the joints and rivets, and a covering of asphalt.

TABLE 4.—WEIGHTS, IN POUNDS PER FOOT, OF LAP-RIVETED PIPES.

Inside diameter, in inches.	Thickness of Plates.							
	⅛ in.	3/16 in.	¼ in.	5/16 in.	⅜ in.	7/16 in.	½ in.	9/16 in.
18	33.9	49.5	64.7	78.6	93.6	109.9	126.7	147.0
20	37.4	54.6	71.0	86.5	102.7	120.5	138.6	160.7
22	41.1	59.7	77.6	94.3	112.1	131.1	150.9	174.4
24	44.5	64.4	83.8	102.0	121.1	141.7	162.8	188.0
27	49.9	72.2	93.7	113.9	135.1	157.8	181.0	208.4
30	55.3	79.8	103.3	125.3	148.7	173.5	199.1	228.8
33	60.8	87.6	113.1	137.2	162.7	189.7	217.4	249.4
36	66.0	95.1	122.3	148.7	176.5	205.4	235.4	269.9
39	71.3	102.5	131.6	160.3	190.4	221.5	253.6	290.2
42	76.6	110.2	141.8	172.2	204.0	237.4	271.5	310.7
48	87.3	125.3	161.0	195.5	231.8	260.3	307.9	351.9
54	98.0	140.6	180.2	218.9	259.2	301.1	344.1	392.8
60	108.8	155.8	199.5	242.5	286.9	333.0	380.4	433.9
66	119.4	170.8	218.7	265.6	314.5	364.9	416.6	474.6
72	130.0	186.1	237.8	288.8	342.0	396.7	452.8	515.6

Table 5 gives the weight per foot of triple-riveted, butt-strap pipe, based on 8-ft. courses, the usual allowance for overweight of rolled plates, the calculated weights of the joints, and a covering of asphalt.

In some of the thicknesses in the smaller pipes, it would be necessary to heat the plates before rolling.

It is desired to emphasize the necessity of placing air-valves on all pipes, as an emergency feature in case the water is suddenly drawn from the pipes. Many calculations have been made on the resistance

to collapsing of pipe against exterior pressure. Such calculations can never take into account all the conditions. The pipe is never a true cylinder, its own weight tends to distort it, and the water inside adds

SAFE STATIC HEAD FOR LAP-JOINT, DOUBLE-RIVETED, STEEL PIPE.
Maximum Stress = 11 200 Lb. per Sq.In. (787 Kg.per Sq.Cm.) Corresponds to a Stress in Plate of 16 000 Lb. per Sq.In. (1125 Kg. per Sq.Cm.) with a Joint of 70% Efficiency.

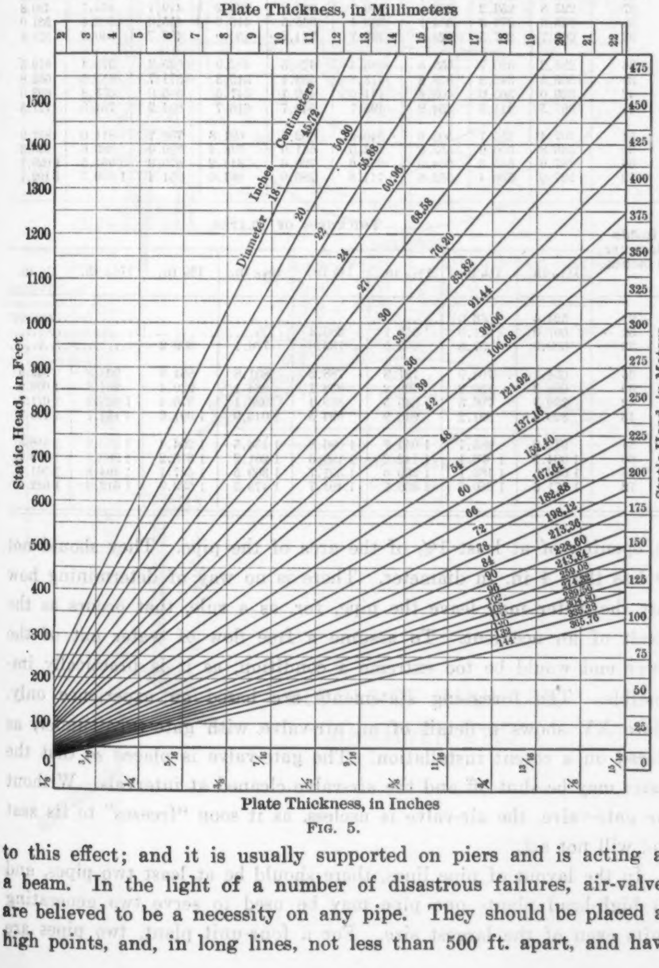


FIG. 5.

to this effect; and it is usually supported on piers and is acting as a beam. In the light of a number of disastrous failures, air-valves are believed to be a necessity on any pipe. They should be placed at high points, and, in long lines, not less than 500 ft. apart, and have

TABLE 5.—WEIGHT, IN POUNDS PER FOOT, OF BUTT-STRAP RIVETED PIPES.

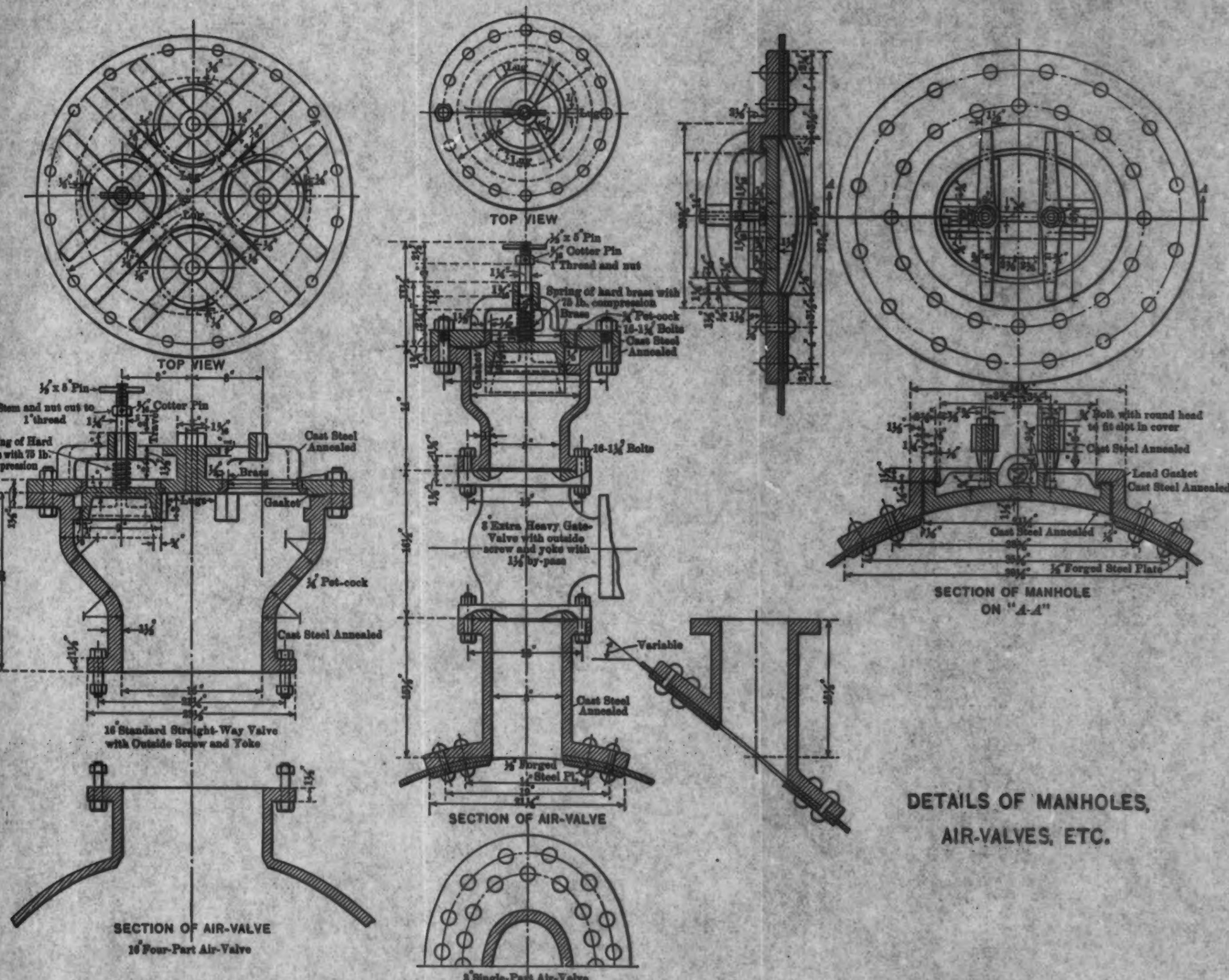
Inside diameter, in inches.	THICKNESS OF PLATES.								
	1/2 in.	9/16 in.	5/8 in.	11/16 in.	3/4 in.	13/16 in.	7/8 in.	15/16 in.	1 in.
24	205.9	230.1	255.7	279.6	307.1	350.3	384.8	426.5	450.3
27	225.8	253.2	280.0	306.7	336.8	384.2	419.1	464.7	490.3
30	245.6	275.3	304.7	333.1	365.5	416.6	455.0	502.4	531.0
33	265.7	297.5	329.3	359.7	394.2	450.0	489.7	540.8	571.8
36	286.2	320.7	353.8	386.4	423.5	482.9	525.3	579.4	612.5
39	305.5	342.3	378.8	413.8	453.4	515.3	571.7	618.5	652.8
42	326.0	365.0	403.6	441.0	482.1	547.6	595.0	655.8	693.0
48	367.5	411.3	454.2	495.7	543.7	616.7	667.9	734.3	777.5
54	407.4	455.7	503.6	549.4	602.6	681.8	738.7	811.0	857.9
60	446.8	500.9	553.8	605.0	661.0	748.4	809.0	886.6	940.3
66	487.6	545.3	604.0	657.6	720.6	814.2	879.9	964.5	1 020.7
72	527.9	589.4	652.6	711.8	780.0	881.0	951.4	1 039.5	1 102.1

Inside diameter, in inches.	THICKNESS OF PLATES.							
	1 1/16 in.	1 1/8 in.	1 3/16 in.	1 1/4 in.	1 5/16 in.	1 3/8 in.	1 7/16 in.	1 1/2 in.
27	524.4	566.0	....	....	....	....	....	....
30	567.0	611.9	653.1	686.4	....	....	....	....
33	609.9	657.8	702.5	736.6	776.1	809.2	....	....
36	654.1	704.9	751.3	788.2	831.8	864.9	908.8	949.9
39	690.3	750.9	799.2	838.1	884.1	920.4	964.8	1 009.9
42	739.3	796.3	846.5	889.4	936.1	976.4	1 025.2	1 071.3
48	826.6	891.2	946.9	994.5	1 046.9	1 091.6	1 144.7	1 195.9
54	912.0	983.7	1 044.7	1 096.8	1 153.5	1 204.4	1 260.9	1 318.5
60	1 004.9	1 076.1	1 142.2	1 199.6	1 261.2	1 318.2	1 380.9	1 443.7
66	1 086.5	1 169.1	1 239.3	1 301.7	1 369.5	1 427.1	1 494.8	1 561.7
72	1 171.6	1 260.6	1 336.1	1 401.7	1 473.5	1 539.9	1 612.9	1 682.4

an opening of at least 1% of the area of the pipe. They should not be less than 4 in. in diameter. There is no way of determining how fast the water may leave the pipe, for, as a rule, that occurs as the result of an accident. To assume a free flow of water out of the lower end would be too extreme a condition, as it is practically impossible. The foregoing statements are based on experience only. Plate XV shows a detail of an air-valve with gate-valve under, as placed on a recent installation. The gate-valve is placed so that the water may be shut off and the air-valve cleaned at intervals. Without the gate-valve, the air-valve is useless, as it soon "freezes" to its seat and will not act.

In the layout of pipe lines, there should be at least two pipes, and on high-head plants one pipe may be used to serve two generating units even of the largest size. For a four-unit plant, two pipes are

PLATE XV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXIX, No. 1347.
GALLOWAY ON
DESIGN OF
HYDRO-ELECTRIC POWER PLANTS.



DETAILS OF MANHOLES, AIR-VALVES, ETC.

sufficient. In low- and medium-head plants, where the quantity of water is large, each generator should have its own pipe. At the junction with the conduit, the pipes should be provided with gate-valves, a cross-over with a gate-valve, and a stand-pipe or multiple-part air-

SAFE STATIC HEAD FOR BUTT-STRAP-JOINT, TRIPLE-RIVETED STEEL PIPE

Maximum Stress = 12 800 lb. per Sq. In. (900 Kg. per Sq. Cm.) Corresponds to a stress in plate of 16 000 lb. per Sq. In. (1125 Kg. per Sq. Cm.) with a joint of 80% Efficiency.

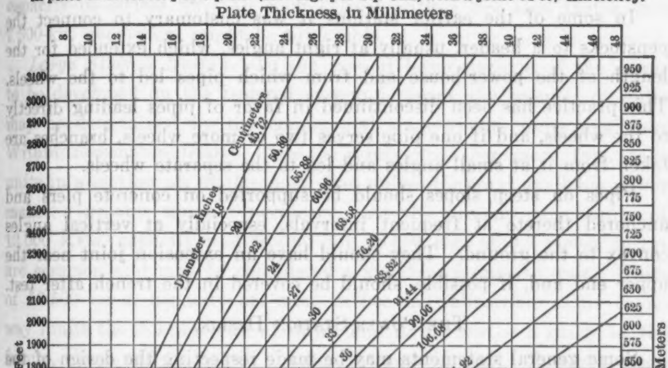


FIG. 6.

valve. The gate-valve should be hydraulically operated, if possible, and controlled from the power-house. On Pipe No. 5 of the Great Western Power Company, water for operating the gate-valve was obtained from a concrete water tank on the hill, 500 ft. above the valve,

water being supplied to the tank from the penstock by an electrically driven pump. At the power-station these pipes should be cross-connected, if very long, so that at off-peak loads, when only a few units are operating, advantage can be taken of all the pipe capacity. Such pipes should be provided with gate-valves at the power-house in such a way that one pipe can be isolated for repairs.

In some of the earlier stations it was customary to connect the penstocks to a header, usually at right angles, which extended for the length of the power-house and from which pipes led to the wheels. This practice has been discontinued in favor of pipes leading directly to the wheels, and if one pipe serves two or more wheels, branches are taken from it at small angles and led to the separate wheels.

Pipes on steep slopes should be supported on concrete piers and anchored thereto at frequent intervals, especially at vertical angles convex to the ground. They should have an extension joint near the upper end and, if possible, should be covered in the trench after test.

The Power-Station Design.

Some general statements may be made respecting the design of the power-station, but in this, as in other parts, the conditions under which the station is to operate will determine the form which the design will take. A station supplying an isolated district differs in layout from one which is merely a part of a large system.

The load factor of the system also has a bearing on the number of units in the station. A high load factor, 50% or greater, would require fewer units than a low load factor, as the more uniform load would not require changes in the quantity of generating apparatus in operation during the day.

Small Station Design.—The number of units in a station of an isolated system should be at least three, of which two would normally carry the load and one be considered a spare unit. If the system is provided with a steam reserve at the receiving end of the transmission line, of a capacity equal to or greater than one-half the full load, then the spare unit at the station may be omitted, if it is necessary to reduce the investment to a minimum. Such considerations give a minimum limit to the number of power units installed. For instance, if the total available power at the peak is 10 000 kw. and the load factor is high, three units of 5 000 kw. each could be installed, two units to be in operation and the other to be considered a spare one.

If the load factor is low, say from 25 to 30%, it would probably be better to install four 3 500-kw. units, three to be in operation and the other to be considered a spare one. The larger amount of installed capacity in the first case, 15 000 kw., would be largely offset in cost by the decreased unit cost of large units as compared with the smaller ones. Such considerations would govern in stations up to a total capacity of 30 000 kw. In such a case, with a low load factor, five 7 500-kw. generators might be preferable.

Large Station Design.—In the case of large stations, if the power to be developed on the isolated system is large, the number of units may be determined by the commercial sizes of the units available. Within reasonable limits, there is rarely need for more than four units in a station or, at the most, five, unless the total available power makes a greater number necessary. At the present time, units of 12 000 k.v.a. capacity and wound for 10 000 kw. at 0.8 power factor are common and may be considered a standard large-size unit. Units of 17 500 k.v.a. capacity are now in use in California. This in no way limits the possible size of units, and if the past is a gauge of the future, there will be a still further increase in size. Two years ago specifications were issued for a generator of 24 000 k.v.a. capacity operated by a 36 000-h.p. maximum rating turbine. Bids were received from responsible bidders for the construction, but it was not carried forward as it was deemed to be too much power in one unit in a station of 60 000 kw. capacity.

It may be said, however, that though there may be no structural or mechanical reason for limiting the size of generators, there may be a limit to the size due to operating conditions and the flexibility of the station. The size of the generator governs the size of the transformers and low-tension switches, and from such viewpoints as these, the concentration of a great amount of energy from one unit may be unwise. Some further consideration of this is given when referring to water-wheel design.

If the 10 000-kw. unit be named as the present standard, the number of units will be a function of the total power available, in any amount above 30 000 kw. If natural conditions reduce the size of unit that is possible, then in large plants there will be a number of the largest possible size. Thus, at Keokuk, Iowa, the plant of the Mississippi River Power Company will contain thirty 7 500-kw. units.

Plants as Part of a System.—The foregoing remarks apply to isolated systems having one generating plant. It was noted that if a steam reserve is included in the system, the spare unit at the hydro-electric station might be omitted. When the contemplated plant will be in a territory where other plants are installed, it is often possible to make reciprocal contracts for emergency power, and this will allow the use of larger units; but this arrangement will hardly warrant the omission of a spare unit.

Where the contemplated plant is part of a large system having a number of generating stations, the unit may be of the largest size available, even to the total of the possible power, and no spare unit is necessary. The Deer Creek Plant of the Pacific Gas and Electric Corporation is an illustration of this plan. With ten hydro-electric plants of 92 000-kw. total capacity and four steam plants of 70 000-kw. capacity on the entire system, this station has one 5 500-kw. generator developing all the available power.

The Limitations of Water-Wheels and Generators.—In the foregoing comment on station design, no account was taken of the limitations imposed by the possible sizes of water-wheels. Two types of water-motors are available for operating generators: the impulse or Pelton wheel, and the hydraulic turbine, generally of the Francis type. The basic conditions of design limit the impulse-wheel to high heads and the turbine to low heads; on medium heads there is a zone where either type is available. The related questions of head, power, and speed control the design of each, and, with the restrictions on design of generators, there are determined certain limits which cannot be passed. All these have a bearing on the size of the unit, and necessarily modify any of the conclusions heretofore expressed.

Generators.—With few exceptions—and those are in Europe—the transmission of energy is accomplished by alternating-current electricity. The number of alternations per second, on different power systems, varies, being now either 25 or 60, with few exceptions, although in earlier work higher numbers of cycles were adopted. The three-phase system can also be said to be general. The number of cycles governs the number of poles and the speed of the generator, and hence, in any one system, the number of available speeds is definitely limited. In the discussion which follows, the 60-cycle system is adopted, as one which is in general use and standard. The pos-

sible numbers of revolutions per minute, within given limits, are as follows: 150, 180, 200, 225, 240, 300, 360, 400, 450, 514, 600, 720, 900, 1800, and 3 600. Owing, also, to the necessity of keeping a practically constant speed on the generator, the head must be reasonably constant. This is readily obtained in high-head plants, but in low-head plants the range of head may be such as to require two wheels on a shaft, adapted to different heads.

General Calculations of Head, Power, and Speed.—In making comparison between water-wheels in different installations, and in the selection of wheels for a projected plant, use can be made of a relation which exists between head, power, and speed. This relation is obtained by developing the equation between unit quantities of head, power, and speed, which gives a result termed the specific speed. The specific speed is defined as that at which a water-wheel would operate if its dimensions were reduced to such an extent that it would generate 1 h.p. at 1 ft. head. The equation is derived in the following manner:

Let H = Effective head,
P = Power of the wheel,
D = Diameter of the wheel,
N = Speed, in revolutions per minute,
H_u = Unit head,
P_u = Unit power,
D_u = Diameter of homologous wheel which will develop unit power under unit head,
N' = Speed of given wheel under unit head,
P' = Power of given wheel under unit head,
N_s = Speed of homologous wheel under unit head and developing unit power, or specific speed.

The velocity of the water (and the corresponding velocity of the water-wheel runner) varies directly as $H^{\frac{1}{2}}$. Therefore,

$$\frac{N'}{N} = \frac{H_u^{\frac{1}{2}}}{H^{\frac{1}{2}}}, \text{ or } N' = N\frac{H_u^{\frac{1}{2}}}{H^{\frac{1}{2}}} \ldots\ldots\ldots\ldots (a)$$

The power developed by any given runner varies directly as the quantity, Q, and the head. Hence, as Q varies as the velocity, which varies as $H^{\frac{1}{2}}$,

$$\frac{P'}{P} = \frac{H_u^{\frac{3}{2}}}{H^{\frac{3}{2}}}, \text{ or } P' = P\frac{H_u^{\frac{3}{2}}}{H^{\frac{3}{2}}} \ldots\ldots\ldots\ldots (b)$$

In the homologous runner of unit diameter, the linear dimensions are directly proportional to those of the given runner, and hence the area of the passages of a turbine, or the jet of an impulse-wheel, vary directly as the square of the linear dimensions. Under a given head, the power varies directly as the area of the water passages. Hence

$$\frac{P'}{P_u} = \frac{D^2}{D_u^{\,2}}, \text{ or } D_u = D\,\frac{P_u^{\frac{1}{2}}}{P'^{\frac{1}{2}}} \dots\dots\dots\dots (c)$$

and

$$\frac{N'}{N_s} = \frac{D_u}{D}, \text{ or } N_s = \frac{D\,N'}{D_u} \dots\dots\dots\dots\dots\dots (d)$$

Substituting in Equation (*d*) the values of N' and D_u,

$$N_s = \frac{D\,N\,\dfrac{H_u^{\frac{1}{2}}}{H^{\frac{1}{2}}}}{D\,\dfrac{P_u^{\frac{1}{2}}}{P'^{\frac{1}{2}}}} = \frac{N\,P'^{\frac{1}{2}}\,H_u^{\frac{5}{4}}}{P_u^{\frac{1}{2}}\,H^{\frac{5}{4}}} \dots\dots\dots\dots\dots (e)$$

when P_u = unit horse-power and H_u = unit head,

$$N_s = \frac{N\,P^{\frac{1}{2}}}{H^{\frac{5}{4}}} \dots\dots\dots\dots\dots\dots\dots\dots (f)$$

This is a general formula for the specific speed, as first developed in Europe. Either the units of the foot-pound-second system or of the metric system can be used, but the numerical quantities will produce different numerical results. As the ratio of the foot to the meter is as 1 : 3.28, and the ratio of the horse-power to the metric horse-power is as 1 : 1.0139, by entering these units in the equation, it can be found that the specific speed metric is 4.447 times that in the foot-pound-second system.* The specific speed furnishes a method of comparison in the design of water-wheel runners, but that subject is outside the province of this paper. In addition, it provides a means

* The writer has followed the method of Mr. Chester W. Larner, in *Transactions*, Am. Soc. C. E., Vol. LXVI, p. 306, in deriving the expression for N_s. There is considerable confusion in the use of this term. It was first applied to the quantity by European engineers, using metric units. Mr. Larner uses the foot-pound-second system of units, but designates the quantity "unit speed". Professor Zowski terms it the "type characteristic" and uses K_t to designate it. Professor Mead, in his work on "Water Power Engineering", uses the square of the quantity, designates it K_s, and names it "specific speed." It seems best to the writer to retain the name "specific speed", even with the use of the foot-pound units, as the relation is a general one and as N is in general use for designating the revolutions per minute, to designate specific speed by N_s. The subject might well be taken up by the American Society of Civil Engineers, and definite recommendations made as to the use of the term.

of comparing installations of water-wheels and discussion of limits of design.

Limitations of Impulse Water-Wheels.—In the development of impulse water-wheels in California, it was soon recognized that a definite relation existed between the pitch diameter of the runner and the diameter of the jet. This relation furnishes a limitation on the amount of power to be derived from one jet, the speed and head being a constant for any one installation. The following discussion is made to determine the limiting value of the specific speed under controlling conditions.

It can be shown that the maximum power is taken from the water when the peripheral velocity of the runner is one-half that of the jet. In practice, the ratio varies from 0.45 to 0.48 V. Experience has also shown that, in continuous operation, the ratio of diameter of wheel to diameter of jet should not be less than 15 to obtain the best efficiency, although less values have been used. In this way, the diameter of the jet establishes a relation between the speed, power, and head, the value of which can be obtained as a maximum value of N_s in the following way,

$N =$ Speed, in revolutions per minute,
$D =$ Pitch diameter of runner,
$d =$ Diameter of the jet,
$\alpha =$ Ratio, $\frac{D}{d}$,
$H =$ Effective head,
$P =$ Power delivered to shaft $= 0.8$ of theoretical power in the water when efficiency of wheel $= 80\%$,
$V =$ Velocity of water at nozzle,
$0.45\ V =$ Velocity of periphery of wheel,
$Q =$ Quantity of water at 62.5 lb. per cu. ft.

Then

$$V = \sqrt{2\,g\,H} = 8.021\,H^{\frac{1}{2}} \ldots\ldots\ldots\ldots(1)$$

$$Q = \text{area of jet} \times \text{velocity} = \frac{\pi\,d^2}{4} \times 8.021\,H^{\frac{1}{2}} = 6.300\,d^2\,H^{\frac{1}{2}} \ldots(2)$$

$$P = \frac{Q \times 62.5 \times H \times 0.8}{550} = 0.573\,d^2\,H^{\frac{3}{2}} \ldots\ldots\ldots\ldots(3)$$

$$\text{Peripheral velocity of wheel} = \frac{\pi\,D\,N}{60} = \frac{\pi\,\alpha\,d\,N}{60} = 0.45\,V \ldots(4)$$

But $0.45\,V = 3.609\,H^{\frac{1}{2}}$ ....(5)

Therefore, $3.609\,H^{\frac{1}{2}} = \dfrac{\pi\,\alpha\,d\,N}{60}$ ....(6)

or, $d^2 = \dfrac{4\,775\,H}{\alpha^2\,N^2}$ ....(7)

By substituting in Equation (3) the value of d^2,

$$P = \frac{2\,725\,H^{\frac{5}{2}}}{\alpha^2\,N^2} \quad \text{(8)}$$

If α is fixed at 15,

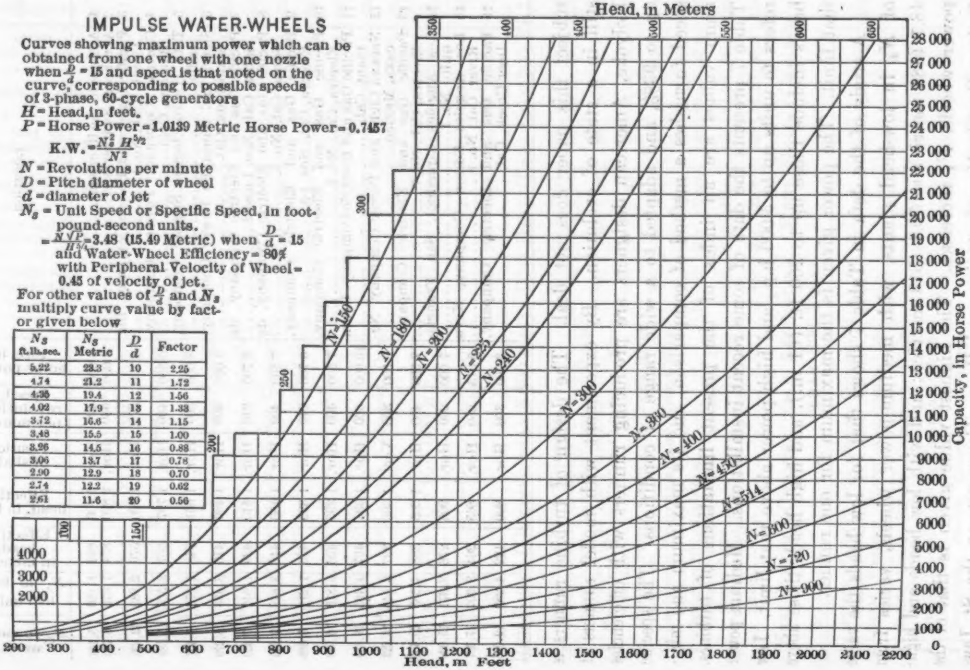
$$P = \frac{12.11\,H^{\frac{5}{2}}}{N^2} \quad \text{(9)}$$

By reference to Equation (*f*), it is seen that the factor, 12.11, is the square of N_s, or $N_s = 3.48$, under the limiting conditions set forth.

Under such conditions, the maximum amount of power to be obtained from any one impulse-wheel with one nozzle can be determined when the head is known. As the speed of electric generators is limited as noted above, it is possible to construct diagrams showing the maximum possible power with different speeds of the unit. Fig. 7 shows a series of curves for ordinary possible generator speeds of 60-cycle current, from 150 to 900 rev. per min. for heads up to 2 200 ft. (670 m.) and power limits up to 28 000 h.p. (26 600 metric h.p.). The efficiency of 80% will represent average working conditions. Factors are also given by which the curve values may be multiplied to change the different values of N_s and $\dfrac{D}{d}$.

It is of interest to note the data of some actual high-head installations, for which purpose Table 6 is given. The horse-power is that of one nozzle on one wheel at maximum rating. At the maximum rating, the value of N_s, in several cases, exceeds that given by Equation (9), namely 3.48. As the normal power is generally 20% less than this, the given values of N_s in Table 6 should be reduced by 10.5% to obtain the value at normal output.

Limitations of Turbines.—The hydraulic turbine is subject to limitations in its design which affect the design of the combined hydro-electric unit. It is possible, however, to vary the design of the turbine runner in such a way that a wide range of conditions can be met and still retain the necessary efficiency in operation. With that



N_s ft.lb.sec.	N_s Metric	$\frac{D}{d}$	Factor
5.22	23.3	10	2.25
4.74	21.2	11	1.72
4.35	19.4	12	1.56
4.02	17.9	13	1.33
3.72	16.6	14	1.15
3.48	15.5	15	1.00
3.26	14.5	16	0.88
3.06	13.7	17	0.78
2.90	12.9	18	0.70
2.74	12.2	19	0.62
2.61	11.6	20	0.56

FIG. 7.

TABLE 6.—DATA ON IMPULSE WHEELS.

No.	Location.	Size, in horse-power.	Speed, in revolutions per minute.	Position of shaft.	Effective head, in feet.	Effective head, in meters.	N_s, in English units.	N_s, in metric units.
1	Adamello. Switzerland	8 000	420	Hor.	2 800	854	1.160	5.160
2	Arniberg, Switzerland	3 000	630	Hor.	2 800	854	1.510	6.720
3	Pacific Light and Power Company, Big Creek, Cal., No. 1	8 750	375	Hor.	1 900	570	2.989	13.320
4	Central Colorado Power Company, Boulder, Colo	10 500	400	Hor.	1 797	507	3.508	15.589
5	Tata Hydro-Electric Supply Company, India	8 000	300	Hor.	1 661	548	1.678	7.467
6	Sierra and Santa Fe Power Company, Stanislaus, Cal	6 000	400	Hor.	1 460	446	3.433	15.270
7	Pacific Gas and Electric Company, de Sabla, Cal., Nos. 1 and 2	3 700	240	Hor.	1 440	439	1.646	7.320
8	Pacific Gas and Electric Company, de Sabla, Cal., No. 3	7 500	400	Hor.	1 450	442	3.872	17.220
9	Pacific Gas and Electric Company, Electra, Cal., Nos 1-5	3 700	240	Hor.	1 395	426	1.211	5.388
10	Pacific Gas and Electric Company, Electra, Cal., No. 6	7 500	400	Hor.	1 395	426	3.816	16.980
11	Pacific Gas and Electric Company, Drum Plant, Cal.	10 000	360	Hor.	1 334	407	4.465	19.850
12	Necaxa Light and Power Company, Necaxa, Mexico	2 500	300	Vert.	1 300	397	3.6464	16.223
13	Pacific Gas and Electric Company, Electra, Cal., No. 7	7 500	400	Hor.	1 205	368	4.583	20.400
14	Los Angeles Aqueduct, San Francisquito No. 1	7 000	200	Hor.	905	276	3.371	15.000
15	Los Angeles Edison Company, Kern River, Cal., No. 1	5 375	250	Hor.	865	264	3.907	17.387
16	Pacific Gas and Electric Company, Deer Creek, Cal	3 700	300	Hor.	800	122	4.289	19.086

subject this paper does not deal. The design of turbine runners is still in a state of evolution. By experiment with model runners at Holyoke, American designers are producing runners with efficiencies up to 93%, and adapted to a wide range of conditions. The specific speed furnishes a method of comparison, and its maximum and minimum values are an index of the present limitations of turbines. Table 7 presents the data of some recent installations. Medium power refers to units up to 5 000 h.p., and high power above that limit. Low heads include those up to 200 ft. (61 m.), and high heads those above that limit. The power given is the maximum for one runner.

A study of the data in Table 7 shows that for low heads the value of N_s in low-head plants, with medium-power wheels, varies from 48.1 (186 metric) to 70.4 (313 metric); and with low heads and high-power wheels, N_s increases. The highest value given is that of the Cedars Rapids Manufacturing and Power Company, on the St. Lawrence River, or 82.3 (366 metric). The value of 75.8 (337 metric) at Keokuk and 75.5 (336 metric) for the 20 000-h.p. wheels of the

TABLE 7.—DATA ON TURBINES.

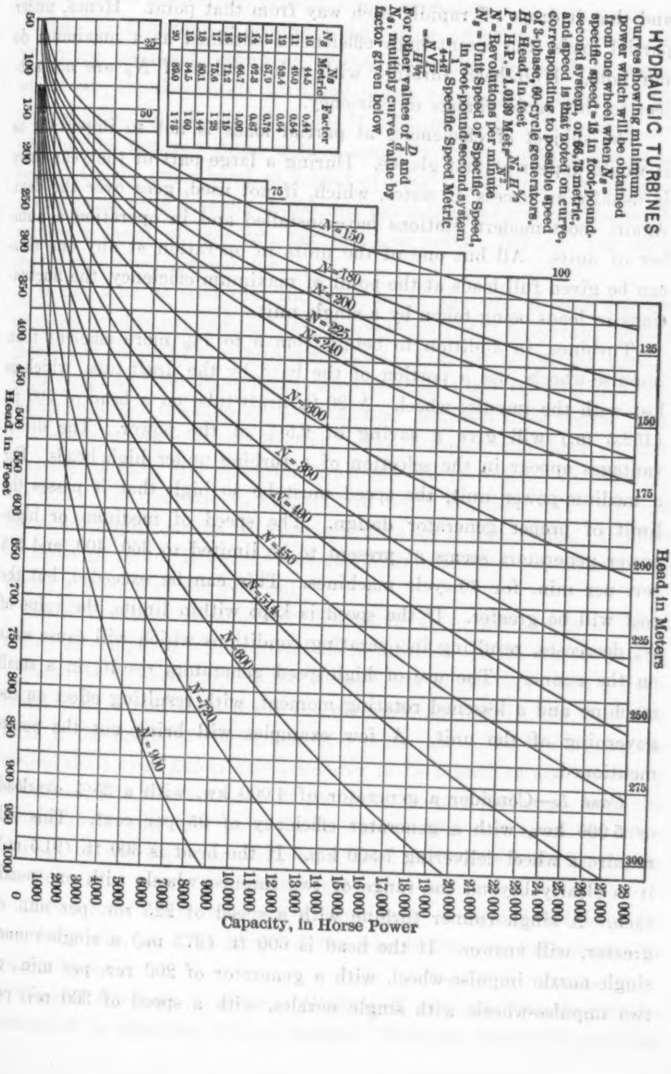
No.	Location.	Size, in horse-power, One runner.	Speed, in revolutions per minute.	Position on shaft.	Effective head, in feet.	Head, in meters.	N_s, in foot-pound units.	N_s, in metric units.
	MEDIUM POWER (UP TO 5 000 H.P.). LOW HEAD (LESS THAN 200 FT.).							
1	Washington Water Power Company, Little Falls	4 500	150	Hor.	66	20.12	52.3	238
2	Appalachian Power Company, Development No. 4	3 500	97	Vert.	34	10.35	69.9	310
3	Georgia-Carolina Power Company, Stevens Creek	3 125	75	Vert.	27	8.23	68.1	303
4	Schenectady Power Company, Schaghticoke	5 000	300	Vert.	146	44.50	48.1	186
5	East Creek Electric and Power Company, Little Falls, N. Y.	4 000	300	Vert.	115	35.05	50.4	224
6	Southern Power Company, Ninety-nine Islands	2 600	225	Hor.	72	21.95	54.7	243
7	Central Georgia Power Company, Ocmulgee River	2 750	300	Hor.	100	30.50	70.4	313
8	East Creek Electric and Power Company, East Canada Creek, N. Y.	4 000	300	Vert.	115	35.05	50.4	224
9	East Tennessee Power Company, Ocoee River	2 700	360	Hor.	98	29.85	60.7	270
	MEDIUM POWER (UP TO 5 000 H.P.). HIGH HEAD (GREATER THAN 200 FT.).							
1	Telluride Power Company, Grace Station	4 250	300	Hor.	450	128.00	9.4	42
2	Cleveland Cliffs Iron Company, Carp River	4 000	720	Hor.	580	176.70	16.0	71
	HIGH POWER (GREATER THAN 5 000 H.P.). LOW HEAD (LESS THAN 200 FT.).							
1	Appalachian Power Company	6 000	116	Vert.	49	14.95	69.3	308
2	Alabama Power Company	17 500	100	Vert.	68	20.75	67.7	301
3	Cedars Rapids Manufacturing and Power Company, St. Lawrence River	10 800	55.6	Vert.	30	9.15	82.3	366
4	Mississippi River Power Company, Keokuk, Ia.	10 000	57.7	Vert.	32	9.75	75.8	337
5	Alabama Power Company, Coosa River Lock No. 12	17 500	100	Vert.	68	20.75	67.7	301
6	Laurentide Power Company, Grand Mère, Canada	20 000	120	Vert.	76	23.20	75.5	336
7	Canadian Niagara Falls	10 000	250	Vert.	130	39.70	57.0	252
8	Central Colorado Power Company, Shoshone Plant	9 000	400	Hor.	170	51.80	61.8	275
9	Washington Water Power Company, Long Lake	11 250	200	Hor.	168	51.20	35.0	156
	HIGH POWER (GREATER THAN 5 000 H.P.). HIGH HEAD (GREATER THAN 200 FT.).							
1	Georgia Railway and Power Company, Tallulah Falls	16 000	514	Vert.	580	176.7	22.8	102
2	Great Western Power Company, Las Plumas, Cal.	18 500	400	Vert.	465	141.7	25.2	112
3	Pacific Coast Power Company, White River, Wash.	18 000	360	Hor.	440	138.2	24.0	107
4	Pacific Gas and Electric Company, Centerville, Cal.	9 700	400	Hor.	565	172.0	13.5	61
5	Michoacan Power Company, Noriega, Mexico	6 000	514	Hor.	670	202.1	11.7	52
6	Seattle Municipal Plant, Cedar River, Wash.	8 000	600	Hor.	584	178.0	18.7	83
7	Tacoma Municipal Plant, Nisqually, Wash.	8 000	450	Hor.	400	121.9	22.5	100

Laurentide Power Company, at Grand Mère, Canada, are next, and represent the most recent installations. It is probable that this limit will be exceeded as development continues, reaching a value of N_s of 90 (402 metric). Such values correspond to relatively low speeds in revolutions per minute, in which case the generator design is susceptible of a considerable variation on account of the large size of the unit.

In this paper more interest attaches to the design of turbines under relatively high heads, namely 200 ft. (61 m.) and greater. Here the value of N_s approaches a minimum. The range is from 9.4 (42 metric) at Grace, Idaho, up to 25.2 (112 metric) at Las Plumas, Cal. To set forth the limitations of turbine design in relation to power and speed, the curves on Fig. 8 were drawn, with a value of $N_s = 15$, the speeds of 60-cycle generators, and a range of head up to 1 000 ft. (305 m.). Opinions may differ as to whether the value of $N_s = 15$ is the proper limit, as a number of plants of higher head have values less than this. It is somewhere near the limit of design, when considering operating conditions, but a table of factors by which to multiply the results shown by the curve is given, by which they can be changed to correspond to other values of N_s.

It is to be noted that the curve values give the minimum quantity of power which a wheel will develop under given conditions of head and speed. To reduce the quantity of power reduces the value of N_s, and this may not be desirable on account of operating conditions, such as injury to the runner from the impact of water under part loads. Hence, at high heads, the problem becomes one where a properly designed wheel may develop more power than required.

Selection of a Water-Wheel.—The selection of the type of wheel for heads varying from 200 to 1 000 ft. (61 to 305 m.) becomes thus a problem involving several factors. The impulse-wheel has a definite maximum of power beyond which it cannot go. Two wheels with one nozzle each represent the maximum power of good design, although more nozzles have been placed on one wheel. The high-power impulse-wheel requires a low speed, with resulting higher cost of generator. For heads greater than 1 000 ft. (306 m.), it can be said that at present it represents the only available type of water-motor, and the many high-head installations are a testimony to its value.



N_s	N_s Metric	Factor
10	44.5	0.44
11	49.0	0.54
12	53.4	0.64
13	57.9	0.75
14	62.3	0.87
15	66.7	1.00
16	71.2	1.15
17	75.6	1.28
18	80.1	1.44
19	84.5	1.60
20	89.0	1.78

FIG. 8.

The turbine, on the other hand, is primarily the water-motor for low heads. The only trouble with turbines with a high value of N_s is that their efficiency is a maximum at a point near the full capacity, and that it drops off rapidly each way from that point. Hence, under partial loads, the recent high efficiencies obtained as a maximum do not hold. High-head turbines with a low value of N_s are not subject to this rapid change of efficiency.

The subject of efficiencies at partial loads is not so important as it would appear at first glance. During a large part of the year there is usually an excess of water, which, if not used, goes over the dam. Again, most modern stations have installed and in operation a number of units. All but one of the units in operation at any one time can be given full loads at the point of maximum efficiency, the fluctuations in loads being taken by a single unit.

Turbines, in addition to being from 5 to 7% more efficient than impulse-wheels, use a portion of the head by the draft-tube, which is lost with the impulse-wheel. A 20-ft. draft-tube on a head of 500 ft. (152.5 m.) will give a saving of 2.5% of the power. The disadvantages appear in the selection of a turbine under high heads. For a medium-power unit, the speed must be so high that it passes the limit of proper generator design. The speed of medium- or high-power generators seems at present to be limited to 360, 400, and 514 rev. per min. for 60-cycle machines. This can be exceeded, but the cost will be greater. If the speed is kept within limits, the value of N_s decreases, resulting in operating conditions which will cause wear on the runner. The use of high-speed generators results in a small machine and a lessened rotating moment, with resulting effect on the governing of the unit. A few examples will bring out the points mentioned.

Case 1.—Consider a generator of 4 000 kw. with a 25% overload, or 5 000 kw., with a generator efficiency of 95 per cent. This will require a wheel delivering 7 500 h.p. If the head is 300 ft. (91.5 m.), it is clearly beyond the range of two impulse-wheels with one nozzle each. A single-runner turbine with a speed of 225 rev. per min. or greater, will answer. If the head is 600 ft. (273 m.) a single-runner single-nozzle impulse-wheel, with a generator of 200 rev. per min., or two impulse-wheels with single nozzles, with a speed of 300 rev. per

min., will answer. Conditions of cost and governing would determine the design. If a turbine were used, the speed would be 900 rev. per min., which is beyond the present limits of design.

Case 2.—Consider a generator delivering 10 000 kw. normally, or 12 500 kw. overload, requiring 18 000 h.p. The lowest head under which two impulse-wheels with one nozzle each, and a speed of 150 rev. per min., would operate is 780 ft. (238 m.). At this head, a turbine of 514 rev. per min., with a single runner, would probably be better. On the other hand, at a head of 1 000 ft. (305 m.) the impulse-wheels could have a speed of 200 rev. per min., and the turbine would require a speed of 600 rev. per min., or better, 720 rev. per min., requiring a special generator.

It can thus be seen that the selection of the size of unit depends on many conditions, and that the choice is restricted to certain definite speeds and limited by certain conditions. For low heads, the turbine holds sway, and for high heads, greater than 1 000 ft. (305 m.), the impulse-wheel is the only available motor. Between these limits, then, is a space where the two overlap. The complex questions of cost, efficiency, design, and operation must also be considered in selecting the unit. The increase in efficiency of units has been dwelt on, and the recent improvements in American turbines is remarkable. It must be borne in mind, however, that the high efficiency can be effective during a portion of the year only.

There has been considerable discussion as to the merits of horizontal- as compared with vertical-shaft turbines. With wheels having a draft-tube to each runner, the accessibility of the unit with a horizontal shaft would seem to be the determining factor. On the other hand, if the floods of the stream rise much above the low water, the vertical-shaft type allows the generator to be placed at any necessary elevation. The free entrance of the water to and discharge from the vertical shaft unit is a point in its favor. The success of the step-bearings of the roller or Kingsbury type, for vertical shafts, makes this feature no longer a deciding factor.

It has been fairly well settled that each runner should have its own draft-tube, as a discharge of two wheels into one tube results in greater losses.

General Remarks on Station Design.—A few other points may be mentioned as affecting station design. Turbines should be provided

with relief valves, of a capacity to discharge at least 75% of the water passing the turbine, and direct-connected to the operating cylinders of the turbine. In the case of impulse-wheels, either the deflecting nozzle or a by-pass nozzle should be used. In both cases, the operation should be directly controlled from the governor.

In a station of several units, there should be, for the governors, a central system of pressure oil supply, with pumps in duplicate and pipes leading to the accumulator tanks at the governors. This system is believed to be preferable to the individual pump at each governor. If the station has vertical shaft units, there should be a similar central system of pressure oil for the step-bearing.

All power units used as auxiliaries should be driven by the primary source of power, the water. The exciter should be in duplicate, and, in addition to the water-wheels, one should have an electric motor to be used in emergencies. The pumps for the oil system can be driven by water, either by piston pumps, rotary pumps, or impulse-wheels. Gate- or pivot-valves back of the turbine should be operated by hydraulic cylinders. The basic reason for these statements is the fact that the auxiliaries are used to control the electric generators. They should be free from trouble at the time when the generator requires attention. If electrically driven, the auxiliaries merely reflect the generator trouble and add to it in emergencies.

All parts requiring cast metal, such as scroll cases of turbines, bodies and bonnets of gate-valves, and curves in pipes subject to water pressure, should be of annealed cast steel. Such parts should be of cylindrical or spherical section. The standard cast-iron gate-valve, with rectangular cross-section of the bonnet ribbed on the outside, is an extreme case of bad design. The ribs should be on the inside, if used at all.

The writer will venture on the electrical design at one point only. If the station is of fair size, say 20 000 kw., there should be a double set of bus-bars, with a complete set of switches for transferring the energy to the lines. Generally, one line takes the output of two or more generators, in which case considerable flexibility is necessary.

In endeavoring to keep within the prescribed limits of the subject matter, the writer may have failed to refer to a number of points which others may consider important. The subject is treated in a

general way, and though definite statements of opinion are advanced, it is recognized that there are many reasons for holding different views. If the paper serves to arouse discussion, especially on the subject of the proper water-motor to use under medium heads, it will have served the purpose for which it was written.

DISCUSSION

Mr. Creager. WILLIAM P. CREAGER,* M. AM. SOC. C. E. (by letter).—The author has said:

"In the design of riveted pipe, a factor of safety of four * * * is sufficient, provided the water-wheels are equipped with by-passes such that pressure rises of more than 25% are not possible."

In penstock design, it is quite customary to base the factor of safety on pressures corresponding to the static gradient. This method, however, as the writer will endeavor to show, is contrary to one of the fundamental laws of economic engineering, namely, that for a structure to be built at the lowest possible cost, all parts must have equal strength; or, in other words, all parts must have the same chance of damage or failure.

The author bases his factor of safety on the assumption that a rise of pressure greater than 25% will not occur. It is evident, however, that before deterioration has taken place, a rise of pressure of about 100% could be accommodated before the penstock has reached its elastic limit or suffered damage. The writer assumes, of course, that in applying this factor of safety of four, under the given assumptions, an allowance for deterioration is being made. The following discussion, therefore, will indicate the conditions which would exist in the various parts of the pipe line, both before and after deterioration, when it is stressed, at the power-house, to its elastic limit.

Referring to Fig. 9, which represents a typical penstock profile: Suppose the penstock is designed to have a factor of safety of four when subjected to pressures corresponding to static gradient. This gradient is indicated, for reference, as "Gradient *A*". It will be readily granted that the only extraordinary loadings to which the penstock will be subjected will be induced by water-hammer incurred by the sudden closing of the terminal gates. A water-hammer gradient, equal, at the power-house, to twice the static head, is indicated in Fig. 9 by "Gradient *B*". Its derivation was based on the assumption that, as the penstock, for this example, is uniform in diameter, the water-hammer above static is approximately proportional at any point to the distance from the reservoir, measured along the center line of the pipe.

In Fig. 10 are plotted the stresses, corresponding to this method of design, in various portions of the pipe, as percentages of its ultimate strength when subjected to pressures corresponding to both gradients, *A* and *B*. That for Gradient *A*, of course, is a horizontal line at 25%, as the penstock was designed for a factor of safety of four based on Gradient *A*. When, however, the penstock at the power-house is

* New York City.

Mr. Creager.

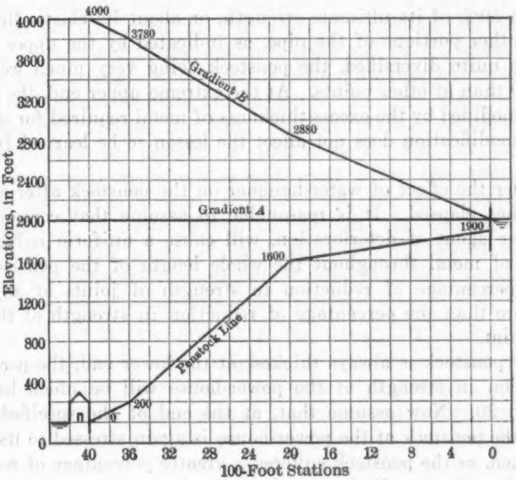


FIG. 9.

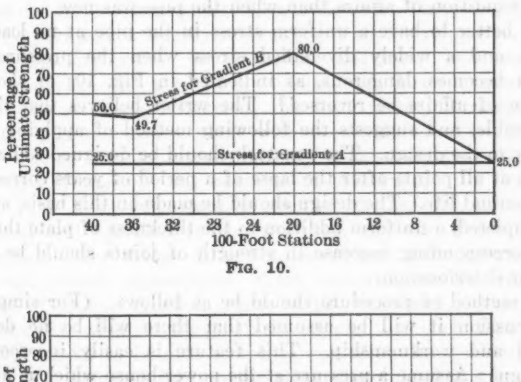


FIG. 10.

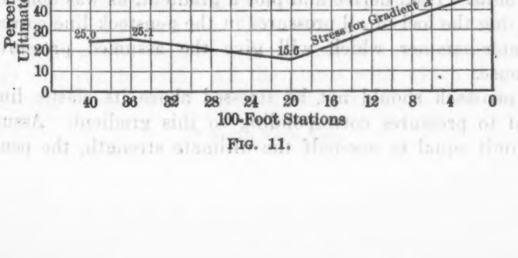


FIG. 11.

Mr. Creager. stressed to 50% of its ultimate strength, or about its elastic limit, the stress in other portions of the pipe, as indicated by the upper line, is seen to be quite diversified, the penstock being very much weaker at Station 20 than at other points. At the extreme upper end, the stresses would be modified by the excess thickness of metal required for stiffness, but such modification does not affect the lesson to be learned from the diagram.

Consider the effect of water-hammer on the penstock after a period of years has elapsed. It is reasonable to assume that corrosion, the most active agent of deterioration, will cause a uniform reduction in thickness of metal throughout the whole length of the penstock, and that the percentage of reduction in strength of joints at any point is not more than the percentage of reduction in strength of the plate at that point.

As the penstock is always thickest at the lower end, the percentage of reduction in strength at the power-house will be much less than at Station 20. Now assume that, at the end of the specified period of years, the penstock at the power-house is again stressed to its elastic limit. Then, as the penstock suffered a greater percentage of reduction in strength at Station 20 than at the power-house, the stress at Station 20, as indicated by Fig. 10, would be considerably augmented, showing a worse condition of affairs than when the pipe was new.

Is it better to have a uniform stress in the pipe at no load on the turbines and a widely diversified stress when the pressure in the penstock becomes dangerous, as indicated in Fig. 10; or should this condition of affairs be reversed? The writer believes that the latter is preferable, and suggests the following method of applying a factor of safety to the design. The penstock should be designed to have equal strength at all points after the lapse of a period of years corresponding to its nominal life. The design should be made on this basis, and, after it is completed, a uniform addition to the thickness of plate throughout and a corresponding increase in strength of joints should be made to allow for deterioration.

The method of procedure should be as follows. (For simplicity in the discussion it will be assumed that there will be no defects in material and workmanship. This feature is easily incorporated in the design): Assume a pressure at the power-house which is absolutely certain not to be exceeded for any condition for which the penstock is to be built. Then derive and plot a gradient, as was done in Fig. 9, representing the loci of all pressures in the penstock line corresponding to a water-hammer which will give the assumed pressure at the power-house.

The penstock should not be stressed above its elastic limit when subjected to pressures corresponding to this gradient. Assuming an elastic limit equal to one-half the ultimate strength, the penstock, at

all points, therefore, should be designed to have a factor of safety of two, based on this gradient. A uniform thickness should then be added, to correspond to the assumed reduction in thickness of metal, and the joint strength should be increased in proportion to the percentage of increase in plate at any point. Mr. Creager.

A diagram similar to Fig. 10, but giving results from the writer's method of design, is indicated in Fig. 11, and represents the conditions which would exist after the assumed deterioration has taken place. In this case, the stress corresponding to Gradient A is quite diversified, though, for Gradient B, when the pipe is on the point of being damaged, the stress is constant at 50% of the ultimate strength, which adheres to the fundamental law of economical engineering previously referred to.

E. NEWMAN,* M. AM. SOC. C. E. (by letter).—This is a very interesting paper, but, in some respects, is not in accordance with the practice which has come under the writer's observation. The author states that the load factor should be determined before the design of the parts. According to the writer's experience, the load factor could not be determined at the time of the installation of the plant, except as a rough guess. The San Joaquin Light and Power Corporation, for example, started with a very low load factor, but the manager has succeeded in building it up to 76% and more. The power developed by this Corporation is largely a pumping load for irrigation, and, by inducing the irrigators to construct reservoirs and use small motors, running all the time, instead of large motors which are idle between each irrigation, the load factor is raised. That it will not pay to build a plant with the load factor too low is, of course, obvious, but, as already stated, it can be improved by good management if there is any business to be secured. In the writer's opinion, it is more important to decide whether the plant can be constructed and operated to compete with gas and Diesel engines; in other words, the things to be considered are: first, interest on the investment; second, depreciation; third, cost of operation; and fourth, a margin of profit. Another thing to be considered is the diversity factor, that is, whether the business in sight is such that one can reasonably expect to be able to sell power after the total capacity of the plant is contracted for; on the same principle that a banker who has large deposits payable on demand, nevertheless, can loan with safety a portion of them, because all the depositors will not call for their money at the same time. The author appears to take the stand that an engineer who designs a plant should know the exact amount of energy that can be sold. All the plants with which the writer has had anything to do were built with the expectation of the business Mr. Newman.

* North Fork, Cal.

Mr. Newman. growing. The water conduits were built to carry the available water, and the reservoir to store as much of the flood-water as possible.

The statement that the number of power units in a small station should be at least three, is misleading, because, although this may be true when there is only one power-house on a system, it is not a fact when the small station is operated in connection with several power stations. In such case, a single unit is the most economical.

The author states, also, that "The size of the regulating reservoir can be determined from the load curve by integration". The size of the regulating reservoir is generally governed by the topography of the country at the head of the penstock, and, in practice, as large a reservoir as it is possible to build, with reasonable expense, is put in.

In regard to conduit design: The writer is of the opinion that the author's values of n (0.014 for concrete, and 0.013 for lumber) are not, under all circumstances, safe coefficients to use in connection with Kutter's formula. The author cites good authorities, and mentions the Floriston flume, where it undoubtedly was a success, but the quality of the water is probably such that slime and barnacles do not accumulate on the sides of the flume to such an extent as has come under the writer's observation.

During the last 12 years, he has been in the employ of the Pacific Light and Power Corporation and the San Joaquin Light and Power Corporation, on the Santa Ana, San Gabriel, Kern, and San Joaquin Rivers, in California. Most of the conduits that convey water from these streams were designed by using the coefficients mentioned by the author, and when constructed, carried the quantities for which they were designed, but after they had been in operation a comparatively short time, it was difficult to get the requisite quantity of water through them, and the writer has superintended the enlargement of conduits on these rivers. From his experience, he has found that, in one instance, a coefficient of $n = 0.017$ should have been used in concrete-lined canals, and $n = 0.015$ in lumber flumes.

On the Santa Ana Canal, the flume, in the shape of a half circle, was built of planed redwood staves. After being in use for a short time, the Company was enjoined from diverting water from the river, and the flume stood idle for about 15 years; it was then sold to the Pacific Light and Power Corporation and used to convey water to a power-house. The writer was in charge of the construction, and soon discovered that the flume would not carry more than 70% of the quantity for which it had been designed, and its sides had to be raised 1 ft. In this instance, the writer was informed that the designing engineer had used a coefficient of $n = 0.011$, but if he had used $n = 0.013$, it would still have been too small. After the reconstructed flume had been in operation for about a year, a section of 32 ft. was destroyed by a rock slide, and was rebuilt with new lumber.

Mr. Newman.

The surface of the water passing through this new section dropped 9 in., and, as the flume was on an even grade, it showed conclusively that the slower velocity in the old portion was due to friction only.

On the Kern River Canal, the Company, at certain intervals, has had to scrape the sides and bottoms of tunnels and flumes in order to pass the requisite quantity of water. The slime and barnacles which form on the inside of conduits carrying water from these rivers vary with the seasons. It is not so bad on the San Joaquin as on the other rivers, and the writer believes that this is accounted for by the higher altitude and resultant colder water of that river.

It is the writer's practice to calculate the theoretical carrying capacity of the conduit according to ideal conditions, and then to add 30% to the cross-sectional area. In some conduits of which he has had charge the moss growth at certain seasons was so heavy that cleaning had to be resorted to, and it would have been impracticable to build a conduit large enough to carry the water without having to resort to cleaning.

As some of the mistakes just mentioned have come to the writer's notice, attention is called to the facts in order that some member may profit thereby; it is not the writer's intention to criticize the author's very able paper.

Mr. Homberger

H. Homberger,* M. Am. Soc. C. E. (by letter).—This paper is to be welcomed from the point of view that it relates to the design of hydro-electric power plants in a comprehensive way, covering the entire field, and giving to the engineer who may be called on to do such work, a guidance in what manner to proceed.

The situation which confronts the engineer who is in charge of designing a power-plant is very well expressed in the author's sentence: "This entire subject becomes one where the conditions surrounding each particular case must govern." In this very sentence is expressed the idea that it is almost futile to lay down fixed laws of design for hydro-electric power-plants. It is one of the characteristics of hydro-electric power work (a characteristic which makes it particularly interesting to the engineer), that it is impossible to generalize or standardize any part of it, and that it very rarely occurs that two problems are alike or nearly so. For this reason, specialists in hydro-electric work have considered it hopeless to write a paper, a treatise, or even a voluminous compendium in which any engineer put in charge of such work could find prescriptions or recipes, which he could pick out and use for the individual case which confronts him. More than in any other line, special preliminary study of each important case is necessary in order to obtain satisfactory results, and both the experience and the ingenuity of the engineer have a great deal to do with success or failure. Any engineer who has specialized in this field, and

* Mill Valley, Cal.

Mr. Homberger. will let hydro-electric power plants built during the last 10 or 15 years pass in review before his mind's eye, will find a sufficient number of cases proving that the above-mentioned contention is justified.

After briefly mentioning the points of reference, Mr. Galloway begins his paper with his conclusions. Taking these up in order, comments on a few of the points will be offered.

The Load Factor.—It is pointed out that the load factor has a very important bearing on the design of the prospective power-plant. In a general way, this is probably correct. Load factors are stated to be from 15 to 25% on a lighting load, 50% on a street-car system, and from 80 to 90% on milling or mining. There are only few power

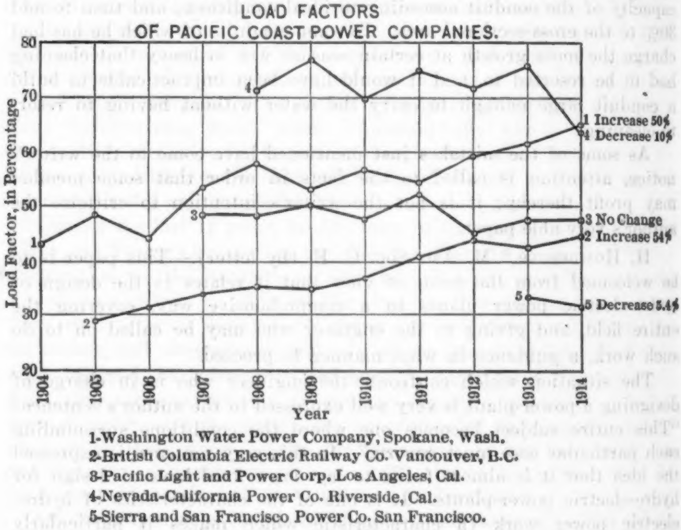


FIG. 12.

systems of medium or large size which can maintain such a well-defined load that they may be put under any of the above-mentioned classifications. It is the endeavor of every power company to obtain a mixed load and to improve the load factor, and the history of the development of the power business shows that often, after a few years, the load factor had become much different from what it had been expected to be when the plant was conceived and constructed. The diagram, Fig. 12, shows the development of the load factors of some of the larger Pacific Coast power companies. It may be seen that in general the tendency of the load factor is to improve, often due to the efforts of the soliciting and commercial departments of the power

companies to invite customers to purchase power at the time of day when the average load is lowest, and also to solicit power consumption in small factories and in homes, a class of business in which a few years ago power companies were less interested. Mr. Homberger.

The load factor of a railway system is variable, and depends largely on the size of the railway system and on the operating conditions. It may be very much higher than 50%, and also considerably lower. Mr. Galloway gives the load factor of the Sierra and San Francisco Power Company as 50 per cent. The principal part of the business of this company is to furnish power for the street cars of San Francisco, with practically no suburban lines. The load factor in 1913 and 1914 was about 30 per cent.

As time goes on, the load of the power company may change entirely in character. Some power companies were established principally to furnish power to mining districts. Mines generally work in three 8-hour shifts all the year around, and consequently furnish a load factor which is very high; but mines, as a rule, have a somewhat limited life. Power companies must plan for a much longer period. In accordance with laws which have been made in the United States during the last 15 years, and others still in the making, one may assume that a power company is to last from 40 to 50 years. When the mining business begins to fail, the power company must look for other markets for its product, and may be very glad to enter into a strictly lighting market, with a load factor of 20%, in order to take off their hands a surplus which it had been marketing under an 80% load factor.

It may be seen from this that the load factor of a prospective power-plant cannot really be foreseen with any close exactness, and is largely guesswork. Unfortunately, this is not a problem of exact science, but is a variable quantity, depending on developments which cannot be anticipated.

The Rules of Economy.—The following comments are offered on the statement:

"In some cases where there is an abundance of water, it is not necessary to strive for the highest efficiency of the various parts, as the losses are not vital."

This is a theory which, unfortunately, has been followed in a number of cases in the design and construction of power-plants. Efficiency is of great importance under any circumstances. It seems to be immaterial how much water may be flowing in a river as long as the entire minimum flow is not required for the power development. It can be shown, however, that to select equipment of low efficiency, on the theory that the water has no value, will lead to the construction of a power-plant which is not economical in operation. Taking it for granted that, for a generating plant of a given size, the efficiency

Mr. Homberger.

of the electric apparatus was fixed by the size and type of apparatus which is offered by manufacturers, let us say that, for a plant of 100 000 kw., 160 000 h.p. was to be developed on the shafts of the water-wheels. Water-wheel makers cannot standardize apparatus like manufacturers of electric machinery, and it is quite likely that the power company will receive proposals for the hydraulic prime movers which vary 10% in the average efficiency guaranteed for an output between 80 000 and 160 000 h.p. in the power-plant. Let us assume that the effective head in the plant was to be 1 000 ft.; that one design of the water-wheels guaranteed an efficiency of 80%, and another one 90 per cent. This would seem to indicate that for the plant the quantity of water which would have to be carried through the pipe lines, head-works, and canal system would vary 12½ per cent. Assuming that the effective head was to be the same for both types of prime movers, it would mean that a pipe of greater capacity would have to be provided for the machine of lower efficiency, and that all the work above the pipe line would have to be correspondingly enlarged. It can be shown that there is only an apparent economy, and that the selection of apparatus of lower efficiency will be very costly, on account of the heavier investment in plant construction. The writer remembers an engineer, who was engaged in the design of a power-plant, making the statement that he had selected turbines, not as much with consideration of their high efficiency, but principally their first cost. That the plant was to contain ultimately four units, at first there were to be two units, and, while the power business was building up and developing, the minimum flow would be twice the capacity of these two units, and that it was, therefore, not necessary to put in wheels of high efficiency. Later, when two additional units would be installed and the entire minimum flow would be utilized, machines of higher efficiency would be selected. This practically means extravagance during the development period of a new business, and economy after the business is established.

Penstock Pipe and Number of Units in a Plant.—Mr. Galloway gives some rules for the number of pipe lines and the number of units in a power-plant. In the layout of pipe lines, the writer cannot see any reason why there should be not less than two pipes; why, in high-head plants, one pipe may be used to serve two generating units even of larger size; why, for four units, two pipes are sufficient; nor why, in a low or medium-head plant, where the quantity of water is not large, each generator should have its own pipe. All these points must be determined by economy of construction and safety of operation. There may be conditions where a high-head plant operates four units from one pipe, though in other cases it may be preferable to equip each individual unit with its own pipe. In many cases it has been economical to use single pipes for a number of units in the upper part

Mr. Homberger.

of the profile, where the pressures are low and a large pipe can be constructed of light material, and to subdivide the line into two or more single lines as the pressure increases. In general, the writer would refer, on this point, also the question of number of units in the plant, to Mr. Galloway's own statement, that the conditions surrounding each particular case must govern.

Regarding the number of units in a plant, the determining factors are: first, the maximum economical size of generating unit that may be constructed; secondly, the factor of safety required for continuous operation of the system; thirdly, whether there are any other hydraulic or steam stand-by plants to help out, making it permissible to reduce the factor of safety of operation in the plant to be designed; and fourthly, the character of the load on the power-plant. The author's conclusion, that the number of power units in a small station should be at least three, and in any station not more than four or five, unless the total available power is greater than can be generated by five units of maximum size, does not appear to be borne out by power-plant practice, either in the United States or abroad.

The Limitations of Impulse-Wheels and Turbines.—The definite relation between pitch diameter of runner and maximum jet diameter has been referred to, and 15 has been given as the minimum for the ratio between the two. Aside from the efficiency, which is affected by this ratio, it may be mentioned that the lower limit is fixed by problems of design. Assuming that the buckets of an impulse-wheel are to be fastened singly to the wheel proper, and the fastening of the buckets is accomplished by bolts traversing bucket and wheel in a direction parallel with the shaft of the machine, it will be readily seen that the necessary bolting space governs the largest size of bucket that may be placed on a wheel of given pitch diameter. The strains on the bolts are dependent on the head under which a wheel of a given size of jet operates; consequently the working head has a direct influence on the lower limit of the ratio and makes it variable. If 15 may be the limit for a head of 2 000 ft., 12 may be satisfactory for a 1 000-ft. head, while, with what is termed a low head on the Pacific Coast, the ratio in extreme cases may be even reduced to 9. The interlocking-chain type of bucket has made it possible to use these low ratios for much higher heads than was possible with the old type of bucket fastening, where each bucket requires two individual bolts. Ratio and specific speed have a constant relationship for all heads, and the following tabulation covers the cases generally occurring with main generating units:

Diameter of jet	1	1	1	1	1	1	1	1	1	1	1	1
Pitch diameter of wheel...	9	10	11	12	13	14	15	16	17	18	19	20
Specific speed.	5.88	5.31	4.77	4.4	4.05	3.76	3.5	3.3	3.1	2.92	2.77	2.64

Mr. Homberger. For exciter units, direct-connected to impulse-wheels, the ratio between wheel diameter and jet diameter often reaches much higher values, so that the specific speed may go even below one.

For Francis turbines, the following tabulation gives a range of specific speeds, where R designates the ratios between the peripheral velocity of the runner and the spouting velocity, as characteristic for runners of different types:

R............	0.585	0.625	0.665	0.7	0.75	0.85
Specific speed...	13.55	20.3	29.4	40.7	56.1	74.8

Both limits of this tabulation have been exceeded in practice; the lowest specific speed the writer knows of is about 10, the highest about 85. Such extremes, however, can only be resorted to with a sacrifice of efficiency, and the very low specific speeds also result in the disadvantage that the runners are bound to wear more rapidly than they do at higher speeds. In order to meet the standard speeds of generators of not too large dimensions, say from 300 to 600 rev. per min., it is necessary to select turbines of high specific speed for low heads, and *vice versa*. A runner of low specific speed becomes large in diameter, and as the area between the runner vanes is proportional to the quantity of water that passes through the turbine, the width of the runner is reduced with increasing diameter. The narrower the passages, the higher the frictional losses, and, at the same time, the wear increases, particularly if the water carries silt. In such cases a higher speed should be selected, even if the generator becomes more expensive on account of special construction; or an impulse-wheel should be resorted to. If the impulse-wheel with single jet cannot furnish the necessary power, two or even more jets may be applied to each wheel, or more than two wheels may be adopted for one unit. The efficiency of such machines is somewhat lower than that of single-jet wheels, but often this is a lesser evil than a Francis turbine, which requires frequent replacement of runners.

In general, it may be said that, for heads ranging from 300 to 600 ft., with specified capacities of the units, the specific speed gives a guidance whether a Francis turbine or an impulse-wheel should be selected. The former will be preferable when the specific speed is greater than 15, the latter when it is less than 5. Between these limits special construction of impulse wheels with multiple runners and multiple jets will generally give more satisfactory results in service than Francis turbines, particularly when high head and water not free from silt prevail.

Mr. Pfau. Arnold Pfau,* Esq. (by letter).—In the introductory of this able paper, Mr. Galloway remarks that very often little consideration of the relation of the machine to other parts of the plant is given, etc. If

* Milwaukee, Wis.

this fact is admitted by a practical engineer engaged in the design of complete plants, what shall we expect of the practice of builders of so-called "home-made" plants. Too much stress cannot be laid on the importance of this very subject, and the writer believes that it is the duty of the engineer, and also to the mutual benefit of the engineer, the manufacturer, and the purchaser, or of the financing interests behind them, to emphasize the importance of proper co-operation of the various parties engaged in the development of water-power with a view of obtaining such results as will guarantee the lasting success of the whole enterprise. Mr. Pfau.

Mr. Galloway has touched on a subject which opens a wide field for discussion, and much remains to be done to raise the efficiency, not only from the engineering, but also (and probably more so) from an ethical, point of view.

There can be no doubt that true co-operation means efficiency.

The true co-operation of engineers or engineering concerns among themselves may be found to constitute a powerful weapon against the condemnable practice of persons lacking either technical abilities or conscientiousness. It may also serve to reduce the practice of "home-made building" of plants by persons not fit to attempt such work themselves, who, however, may have been justly forced to such a procedure on account of disappointments experienced before, with engineers lacking professional ability or scrupulousness.

The true co-operation of the manufacturers among themselves would serve to prevent abuse of their confidence on the part of prospective purchasers or engineers. Such abuse exists in cases where manufacturers are requested to furnish elaborate plans, specifications, and guaranties, when these are only intended for use in preparing a report on a proposition which may be justly called nothing more than a "wild-cat scheme." Is it not "abuse" when leading manufacturers are requested to expose their engineering and prepare plans and specifications which will be used by the purchaser as a basis for final specifications which are then sent to manufacturers who themselves are incapable of designing what has been collected by the engineer and has been combined as the best solution of the problem? Is it not an injustice to the conscientious client to make him carry the manufacturer's burden of the general expenses caused by parties who are neither connected with him nor friendly toward his purveyor? Co-operation, also, along the lines indicated, would have as beneficent an effect as co-operation in regard to technical matters. Space does not permit one to go any further here. If the remarks made serve to stir up the sentiment, they have answered the purpose for which they were brought forth.

Mr. Galloway states that the turbine designer may set forth requirements which the generator builder cannot meet. The writer's experi-

Mr. Pfau. ence leads him to believe that it more frequently happens just the other way; that is, that the generator builder has fixed conditions which, normally, cannot be met by the turbine builder. It is only a few years ago that the practice existed first to look for a standard generator which could be obtained at a favorable price—if not second-hand—and then force the turbine manufacturer to build a machine which would meet the sometimes very freakish requirements of power and speed. Even now a generator is often purchased with a fly-wheel effect of its rotor insufficient to secure satisfactory speed regulation of the unit, although, with little extra expense, such fly-wheel effect could be most efficiently built into the rotor without impairing the efficiency of the generator.

A hydraulic prime mover, like no other prime mover, is dependent on the manifold characteristics of its operating fluid supplied by Nature. It is the head and flow which are fixed with each development; it is the quality of the operating water, the climate, and many other factors which enter into consideration in the selection of the proper size and type of turbine; and it is natural that the specialist should know most about it, and, therefore, he should be given all the underlying data for deciding on the best selection.

Under the heading "Load Factor" Mr. Galloway suggests operating the steam plant over the peak and raising the load factor on the hydro-electric plant. The writer invites attention to one phase of this condition: Suppose it costs more to produce a steam, or other than a water-power, kilowatt, and suppose the water-power is combined with a storage reservoir; then it would seem to be more economical to let the water-power plant take care of the load fluctuation and set the governors of the steam plant so that the steam-power generating unit will operate at the point of highest economy. This can be done by providing a load-limiting device, and a relay on the governors of the steam-power units, which is set so that it does not allow the power to be changed beyond the best economy point in either direction, except in case the load demand should (through short circuit or other disturbance) become less than the power produced by the steam (or other prime mover) unit. Consider, for instance, an electric-railway system operated jointly by steam and storage water-power prime movers. The load factor of such a system may be very low; however, all the stand-by capacity must be ready at a moment's notice to take care of sudden peaks. It would certainly be most economical to let the steam plants always carry the minimum load, so that these units would be prevented from operating at a point of commercially low economy.

A splendid and welcome remark has been made by the author under the heading "The Value of Power", namely: that if money is to be spent, it is much better to apply it at the receiving end. It is not because the writer belongs to the manufacturing group that he points

to the importance of the remark "at the receiving end", but more because, in his experience, he can cite many cases where the financial success of an enterprise has been seriously hampered by neglect of this very question. Mr. Pfau.

Thousands and hundreds of thousands of dollars are spent on hydro-electric developments. The most splendid engineering and construction work is carried out, and nothing is left undone to render the construction work perfect. How different is sometimes the procedure in the selection and purchase of the machinery; how little money is left for the prime mover, the most vital factor of the whole enterprise, and the one which converts the natural resources of power into the "dollars and cents" which are intended to flow into the pockets of the owners. It is either the lack of adequate financial resources, or too extravagant construction work, or the lack of appreciation of the importance of properly selecting and purchasing the prime-mover equipment that creates so many hydro-electric plants in which the prime mover constitutes the weakest and least dependable link of the whole system. To illustrate this statement the writer will cite a remark made by the operating manager of one of the larger hydraulic plants in the United States: "If we had known before what we know now, we could have accepted the highest bid, paid the bidder extra for nickel plating every nut on his equipment, and still be money ahead, because we would not now have the continual expense of up-keep and replacement". The truth of this statement is acknowledged when one looks at the graveyard of parts out of commission stored near his power-house.

Under the heading "Generators" Mr. Galloway cites the possible numbers of revolutions per minute for 60-cycle generators. The speed of 150 rev. per min. may be considered low for medium heads. Single-runner, vertical-shaft units operating under low head, however, may operate under considerably lower speeds, and, therefore, synchronous speeds of 120, 100, 90, 80, 72, 60, and $57\frac{1}{2}$ may be justly added. In the early Nineties a low-head plant was built in Switzerland with vertical-shaft, Jonval turbines, directly connected to alternating-current generators at 28 rev. per min. These generators, however, did not satisfy, and were replaced by horizontal-shaft generators, of double capacity, operating at 56 rev. per min., and had beveled mortise gears, driven from the vertical shafts of two turbines each. It should be borne in mind that low-speed generators have a lower efficiency than high-speed machines of equal capacity. This disadvantage, however, may be more than offset by the increased efficiency of a single runner of large diameter over that of a plurality of small runners, and it may also be offset by advantages of simplicity of design and better maintenance of a single, vertical-shaft arrangement with outside (accessible) gate rigging, etc.

Mr. Pfau. A more detailed discussion of this subject may be found in the writer's paper on Topic No. 20, to be presented before the International Engineering Congress, 1915, at San Francisco.*

Under the heading "Limitations of Impulse Water-Wheels", Mr. Galloway states that experience shows that the ratio of wheel diameter to jet diameter should not be less than 15, in order to obtain the best efficiency. (Reference is also made to the term "pitch diameter", but the writer would suggest the adoption of the term "impact diameter", being the diameter of the wheel circle tangent to the axis of the jet.) The ratio given represents a very safe figure. There are many factors, of course, which determine the best efficiency. Some eight years ago this was a subject of acute discussion among engineers connected with the design of buckets, but now it has been generally acknowledged that the angles and the number of buckets greatly influence the question of best efficiency. The term "specific speed" is applicable to impulse-wheels as well as reaction, or other, turbines. It explicitly represents nothing else than a ratio, as follows:

For impulse-wheels.................. $N_s = C \dfrac{d}{D}$;

For turbines.................. $N_s = C^1 \sqrt{\dfrac{B}{D}}$;

C and C^1 being constants of fixed value for a fixed design only.

The explicit value of C is:

$$C = \frac{60 \times 2\, g \sqrt{2\, g} \sqrt{\dfrac{\pi}{4}} \sqrt{62.5}}{\pi \sqrt{550}} \times U^x \times \alpha \times \sqrt{e_j\, e_b}$$

in which U^x = peripheral speed coefficient at impact diameter;

$$U^x = \frac{D\, \pi \times (\text{rev. per min.})}{60 \sqrt{2\, g\, H}};$$

α = the coefficient of velocity of the jet;

$$\alpha = \frac{Q \times 4 \times 144}{\pi\, d^2 \sqrt{2\, g\, H}}, \text{ if } d \text{ is in inches};$$

e_j and e_b = the efficiency of the jet and the buckets, respectively; and α and e_j are related values;

$$e_j = \frac{M}{2} \left(\alpha \sqrt{2\, g\, H}\right)^2$$

$$\frac{M}{2} \left(1 \sqrt{2\, g\, H}\right)^2 = \alpha^2.$$

From the foregoing it can be seen that the specific speed is a function, not only of the ratio of jet and impact diameter, but also of

* As this paper has not yet appeared, it has been found inopportune to publish some of the data here.

the peripheral speed of the wheel, the efficiency of the jet, and of the buckets, the latter being influenced naturally by the discharge losses, consequently by the relative discharge angles of the buckets. It thus follows that considerable flexibility of the characteristic can also be obtained with an impulse-wheel by properly selecting its fundamental values embodied in the design. **Mr. Pfau.**

A discussion along these lines was first opened in connection with the design of the impulse-wheels of the Kern River Plant No. 1, of the Southern California Edison Company.* The correctness of the design is proved, inasmuch as these buckets are still in operation and show absolutely no signs of corrosion, in spite of the fact that heavy duty is imposed on them due to the continuous partial impingement of the jet issuing from a governor-controlled deflecting nozzle.

The values of specific speed applied to turbines are still more flexible than those applied to impulse-wheels, and, therefore, a very wide range is found in practice. With turbines, the term "specific speed" also represents the product of a constant multiplied by a ratio. The ratio is in some respects similar to that of the impulse-wheels, inasmuch as for the jet diameter is partly substituted the clear height of the runner. The value of the constant, C^1, is explicitly:

$$C^1 = \frac{60 \times 2\,g\,\sqrt{2\,g}\,\sqrt{\pi}\,\sqrt{62.5\,U^x}\,\sqrt{V_3{}^x}\,\sqrt{e}}{\pi\,\sqrt{550}}$$

in which U^x represents the peripheral speed coefficient of the runner at the entrance; $V_3{}^x$ is a radial velocity component, or that velocity coefficient which is figured from the formula:

$$V_3{}^x = \frac{Q}{D\,\pi\,B\,\sqrt{2\,g\,H}};$$

Q being the discharge, in cubic feet per second, D the average entrance diameter, and B the vertical height of the runner. e is the total efficiency of the turbine. As the values, U^x and $V_3{}^x$, vary throughout a very wide range, it is evident that, for a fixed efficiency, e, the specific speed of a turbine also may vary throughout a very wide range. Another low specific speed of a successful turbine is 12.8 (English) or 56.8 (metric). It is a replacement of a Girard (or action) turbine by a Francis (or reaction) turbine, whereby head, power, and speed were fixed by the conditions of the existing unit.

The highest specific speeds are fixed by the progress of the art in designing high-speed, high-power runners, although they are applicable only to low heads. Specific speeds exceeding the value, 100, are in successful commercial operation. Small model runners have

* The diagrams were published (although on a rather distorted scale) in *Engineering News*, December 24th, 1908.

Mr. Pfau. been tested with specific speeds as high as 207 (English) or 920 (metric), and with very satisfactory efficiencies.

A curve has been given in the paper previously mentioned showing the specific speed as a function of the head, from which it may be determined, and an empirical formula will also be found from which specific speed can be computed for a given head, or *vice versa.*

Mr. Galloway correctly states that trouble is invited with the adoption of a specific speed too low for high-head Francis turbines; such runners have a tendency to operate as non-ventilated, action turbines. The failure of admitting air under such conditions is responsible for the corrosion of both runner and guide-case, and in many cases also for vibrations and hydraulic noises.

One more discussion may be in order in connection with head, in the selection of a water-wheel, and particularly with regard to the question of utilizing the draft-head. Draft-tubes, however, have been applied also to impulse-wheels, with more or less success. If correctly applied (which can only be done in connection with a float controlling an air-valve by which the level of the discharge water is kept within a certain distance of the lowest point of the wheel buckets), negative pressure is formed in the wheel housing, according to the height of the suction column. This negative pressure accelerates the jet issuing from the nozzle, and thus virtually increases the effective head accordingly. It can be readily seen, however, that the velocity of the water leaving the buckets is not reduced, and consequently its energy is not recovered.

Not so with the draft-tube applied to a reaction turbine. There it is not only the additional suction head that is utilized by its application, but additional energy is obtained by the retardation of the water column discharging from the runner. This is particularly important in connection with low heads, where the suction head may constitute a large percentage of the total available net head, and more so with high specific speeds, where the velocity of the water leaving the runner is relatively high ($0.45\ \sqrt{2\,g\,H}$ or more), which would seriously impair the efficiency of a turbine, if not carefully reduced by a properly designed draft-tube.

The writer can cite a case where the output of a standard stock turbine was increased 34% by no other change than that from a short, stub draft-tube to a concrete draft-tube of proper design. It is advisable, therefore, with low-head developments, to hold the turbine manufacturer responsible for the dimensions of the draft-tube.

Mr. Galloway's paper should receive the best of study, and the many points mentioned by him should be used liberally for discussion and further explanation. This would serve to unite better the parties connected with the development of water-power, from engineering, as well as social or ethical, points of view.

J. D. GALLOWAY,* M. AM. SOC. C. E. (by letter).—Several who have discussed the paper have referred to the subject of load factor. It is well known that a load factor can be built up by good management, and Mr. Newman has noted how that of the San Joaquin Light and Power Company was increased from a relatively low one to one of 76%, which is relatively high. Mr. Homberger has also remarked on the increase of load factor. In earlier times, this could only be guessed at, but it would seem that, with the experience of many companies available, a much closer approximation to the probable load factor could be made. An engineer with Mr. Newman's years of experience could now go into new territory and make a fair estimate of a possible load factor. Mr. Galloway.

Mr. Homberger questions the statement concerning the given load factor of the Sierra and San Francisco Power Company. The data of the street-car system of San Francisco were given to the writer by the General Manager, and are believed to be correct.

Mr. Pfau has expressed a belief that where a kilowatt-hour produced by steam costs more than one produced by water-power, the steam units should be allowed to operate continuously at or near the most efficient point, the load variations being taken by the water-power plant. The writer is not prepared to say that this is not true in some cases, possibly where the two plants are near each other. However, when the transmission line is long, it is believed that the steam plant should only be used to take the peak and the water plant allowed to produce the major part of the energy. The investment in plant is much reduced, as noted in the paper, to which the reader is referred.

Mr. Newman makes some valuable comments on the value of n in Kutter's formula, and cites data based on experience. His allowance for contingencies in the design of conduits is quite to the point. Most engineers will add something to the size of a conduit after designing it by formula. As to the design of the regulating reservoir, Mr. Newman overlooked the writer's statement that, although the reservoir capacity can be obtained by integration from the load curve, it should at least hold water for one day's run or more, if circumstances will permit.

The number of units in a plant was commented on by Mr. Homberger and Mr. Newman. The latter overlooked the writer's statement that the small plant with three units is an isolated plant. It was noted in the paper that when such a plant is part of a large system, one unit is sufficient, and the Deer Creek Plant of the Pacific Gas and Electric Corporation was cited as an example. Mr. Homberger states that the general rules given as to the number of units are not borne out by the design of plants in the United States or in Europe. The writer referred to modern plants. Earlier plants, which were subject to the changes of evolution, are not examples. Such plants as the

* San Francisco, Cal.

Mr. Galloway.

Drum Plant or the two Big Creek Plants, in California, both recently constructed, are good examples. In both, the number of generators was kept low and the size was increased above those formerly used. At the Drum Plant four 12 500-kw. units form the ultimate installation. If the plant had been one of an isolated system, possibly the designer would have added another as a spare unit. Under the conditions, the plant being part of a large system, four very large units were advisable. Mr. Homberger would hardly advise the use of ten 6 000-kw. units, or some other similar combination, for this plant.

The writer's reasons for the statements regarding the number of pipe lines are as follows: Two pipes are to be preferred to one. If an accident happened to the single pipe, the entire plant would have to be closed down during repairs. Two pipes will allow one to operate nearly all the plant during repairs to the other. Not more than two units on one pipe were advised, for the reason that an injury to the pipe would shut down too much generating capacity. Individual pipes to each generator in low and medium-head plants were advised because the quantity of water is great, and cross-connections, elbows, or Y's are difficult to construct. The theoretical quantity of metal in one large pipe is equal to that in two smaller pipes of the same total capacity.

The writer differs from Mr. Creager in the assumption as to the pressure in a pipe from surges varying uniformly from a maximum at the power-house to zero at the upper end. Experience at two power stations shows that the increase of pressure near the top of the pipe is almost the same as that at the bottom, as recorded by gauges. The writer believes that it is not well to design a water-wheel operating under a head of 2 000 ft., the closing of the gates of which may cause an increase in pressure over that of the static head equal to 100 per cent. It would be a waste of money in the pipe and also a dangerous condition in the unit. Such a unit would require an impulse-wheel. If it is provided with a deflecting nozzle or proper by-pass, the water column is not checked by governor changes, and little or no pressure rise results. Relief valves on turbines are not quite as efficient, but modern ones are built, which keep the surge pressures far below the quantity cited.

Mr. Homberger questions the statement that during a great part of the year the efficiency of the water-wheel is not important. It was not intended as a reason for buying a machine with an efficiency of 80%, when one of 90% was available. The basis of the argument can be illustrated by an example: Assume a 10 000-kw. generator, one of a number in a power-house. A difference in efficiency of 1% represents 100 kw., equivalent to 876 000 kw-hr. per year, which, at a value of ½ cent per kilowatt-hour, represents a difference of $4 380. Capitalized at 6%, this represents $73 000 as the value of the 1 per

cent. The amount is greater than the total cost of the hydraulic unit. Obviously, the purchaser could not pay that much additional for the 1%, for the reason that, during a great part of the year, no returns could be had on the extra 1 per cent. A little more water through the less efficient wheel will produce the maximum output from the generator. Throughout most of the year, this water is available and, if not used, goes over the dam and is wasted. Such considerations reduce materially the value of the saving of a small percentage in the efficiency of wheels. However, this in no way should lead to the selection of a low-efficiency unit. Mr. Galloway.

The remarks, by several of those engaged in the manufacture of water-wheels, as to the limiting values of the specific speed are to be taken as authoritative. The writer was not in a position to offer any but general data on this subject, based on given installations. The paper mentioned by Mr. Pfau, which is to be presented before the International Engineering Congress, 1915, will be a considerable addition to the literature on this subject. The extreme limits of specific speed given for some installations cannot, however, indicate ordinary limits of good design. This is given rather by the general formula or by definite statements.

The writer desires to express his appreciation of the kindness of those who have taken the trouble to discuss the paper.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 1348

COMPUTING RUN-OFF FROM RAINFALL AND OTHER PHYSICAL DATA*

By Adolph F. Meyer, M. Am. Soc. C. E.

With Discussion by Messrs. Joel D. Justin, A. M. Strong, E. F. Chandler, C. E. Grunsky, Robert E. Horton, W. G. Hoyt, and Adolph F. Meyer.

Synopsis.

The paper points out the necessity of basing conclusions with respect to stream flow on extended physical data. As most stream-flow observations available for the engineer's use extend over a comparatively few years, there is need for a method of computing run-off from other physical data for the purpose of extending and supplementing short-term stream-flow records.

Inasmuch as run-off or stream flow consists of the residual rainfall after all losses have been deducted, the paper treats first of the method for computing these losses.

The loss of rainfall through evaporation from water surfaces and from snow and ice is first discussed. The factors modifying the rate of evaporation and their relative importance are considered, and curves are presented, based on observations and checked by an evaporation formula, for the computation of these losses for any given water-shed, from temperature records, and the area and character of the bodies of water found on the water-shed.

Evaporation from land areas is next discussed, with respect to its variation with temperature, season, rainfall, vegetal cover, topography,

* Presented at the meeting of April 21st, 1915.

soil, and subsoil, and curves are presented for the determination of the loss of rainfall through evaporation from land areas for various temperatures and rates of rainfall. The values taken from these curves are to be modified by the use of certain coefficients based on water-shed characteristics.

The losses out of rainfall resulting from the transpiration of plants, as determined by various investigators, are briefly mentioned, and some of the underlying principles governing the transpiration of plants are pointed out. The water requirements of plants are discussed, and a curve is presented to aid in computing monthly transpiration losses.

A summary statement of the author's method of computing run-off is next made, and the method applied to fifteen widely different water-sheds. A brief statement of the outstanding physical characteristics of these water-sheds is given, to aid the reader in understanding the choice of coefficients used in the computations.

The variation of evaporation and transpiration losses for various types of rainfall distribution exemplified by the given water-sheds is discussed, and the necessity of taking monthly temperatures into consideration in every method of computing evaporation and transpiration losses is pointed out.

A summary is given of the physical data and computations for the fifteen water-sheds discussed, together with detailed computations for characteristic years. The water-sheds used range in area from 100 to 37 000 sq. miles, with a variation in average annual rainfall of from 14.8 to 61.9 in.; a variation in mean annual temperature of from 37 to 66°, and a variation in mean annual run-off of from less than 1 in. to more than 50 in. Reasonably close agreement between computed and observed annual run-off under these widely varying physical conditions is shown in every instance.

The monthly distribution of run-off is discussed, curves are presented, and the author's method is applied in detail to the Root River water-shed, in Minnesota.

To show the variation in annual rainfall and run-off, and to emphasize further the need for long-term records, two stations are selected and curves presented, giving 5-year progressive mean values and cumulative mean values of rainfall and run-off. On one of the

water-sheds selected, the 5-year mean run-off, for example, varies from 66 to 142% of the 17-year mean run-off.

INTRODUCTORY COMMENTS.

From time to time various curves and formulas designed to give the annual yield of water from any given water-shed, and its distribution throughout the year, have been presented. Perhaps the most common expression of these quantities has been in terms of percentage of precipitation. Whenever this method has been adopted, great variations in run-off for the same quantities of precipitation have been noted. In fact, the lack of direct relationship between rainfall and run-off is a fact of common observation among those who have made a study of such data. Run-off, for a given month, considerably in excess of the rainfall for the same month, is not an exceptional occurrence on many streams of the country. For the same annual rainfall the annual run-off occasionally varies by nearly 100% on the same stream.

These facts are well illustrated by Fig. 1.* The run-off for April, in Fig. 1, shows a variation of from 5 to 200% of the rainfall on the same water-shed. Moreover, the high percentage is for the lower rate of rainfall. The run-off for September shows a variation of from 2 to 140% of the rainfall for practically the same precipitation on the same water-shed. The annual rainfall for one of the streams on Fig. 1 varies from 6.7 to 11.97 in., or about 80% for practically the same annual rainfall.

In attempting to express the relationship between rainfall and run-off, Vermeule† used a constant plus a percentage for the various months of the year, and varied this relationship on different water-sheds with the mean annual temperature.

Justin‡ expressed annual run-off by an equation consisting of a coefficient (which varied with slope and mean annual temperature for different water-sheds) multiplied by the square of the annual rainfall.

Babb§ used curves giving the monthly run-off to be expected from any given water-shed in the various parts of the country, in terms of

* Compiled from "The Flow of Streams and the Factors That Modify It," by D. W. Mead, M. Am. Soc. C. E.

† Water Supply of New Jersey, 1894; and Annual Report, State Geologist of New Jersey, 1899.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 346.

§ *Transactions*, Am. Soc. C. E., Vol. XXVIII, p. 323.

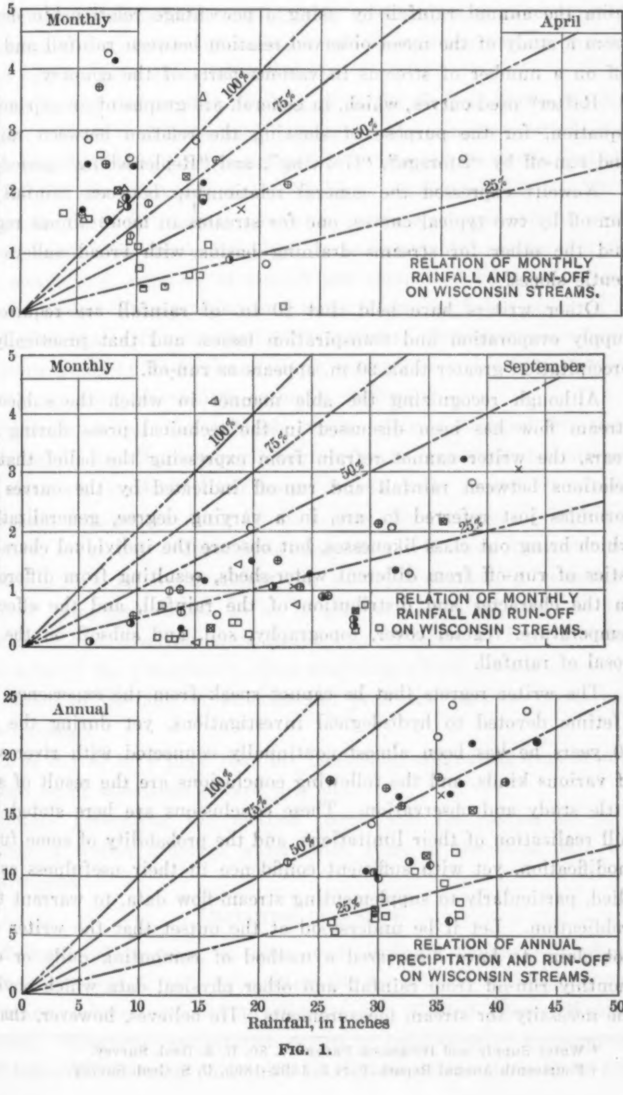

Monthly
April
100%
75%
50%
25%
RELATION OF MONTHLY
RAINFALL AND RUN-OFF
ON WISCONSIN STREAMS.
Run-off, in Inches
Monthly
September
100%
75%
50%
25%
RELATION OF MONTHLY
RAINFALL AND RUN-OFF
ON WISCONSIN STREAMS.
Run-off, in Inches
Annual
100%
75%
50%
25%
RELATION OF ANNUAL
PRECIPITATION AND RUN-OFF
ON WISCONSIN STREAMS.
Run-off, in Inches
Rainfall, in Inches

FIG. 1.

a percentage of the total annual run-off. The latter was computed from the annual rainfall by using a percentage relationship derived from a study of the mean observed relation between rainfall and run-off on a number of streams in various parts of the country.

Rafter* used curves, which, in general, are graphs of an exponential equation, for the purpose of showing the relation between rainfall and run-off by "Storage", "Growing", and "Replenishing" periods.

Newell† expressed the general relationship between rainfall and run-off by two typical curves, one for streams in mountainous regions and the other for streams draining basins with broad valleys and gentle slopes.

Other writers have held that 20 in. of rainfall are required to supply evaporation and transpiration losses, and that practically all precipitation greater than 20 in. appears as run-off.

Although recognizing the able manner in which the subject of stream flow has been discussed in the technical press during past years, the writer cannot refrain from expressing the belief that the relations between rainfall and run-off indicated by the curves and formulas just referred to, are, in a varying degree, generalizations, which bring out class likenesses, but obscure the individual characteristics of run-off from different water-sheds, resulting from differences in the character and distribution of the rainfall, and the effect of temperature, vegetal cover, topography, soil, and subsoil on the disposal of rainfall.

The writer regrets that he cannot speak from the experience of a lifetime devoted to hydrological investigations, yet during the past 10 years he has been almost continually connected with river work of various kinds, and the following conclusions are the result of some little study and observation. These conclusions are here stated with full realization of their limitations, and the probability of some future modification, yet with sufficient confidence in their usefulness as applied, particularly to supplementing stream-flow data, to warrant their publication. Let it be understood at the outset that the writer does not claim to have discovered a method of computing daily or even monthly run-off from rainfall and other physical data which obviates the necessity for stream measurements. He believes, however, that he

* Water Supply and Irrigation Paper No. 80, U. S. Geol. Survey.

† Fourteenth Annual Report, Part 2, 1892–1893, U. S. Geol. Survey.

has found a method of computing the annual run-off from widely different water-sheds with considerable accuracy, and of computing a reasonable distribution of such run-off through the various months of the year for most of such water-sheds.

Considering the number of streams in the United States the discharge of which is of industrial importance, the number of stations at which stream flow is being measured is comparatively small, and the periods for which records are available are relatively short. If it takes from 30 to 40 years to secure an accurate measure of the mean annual rainfall at any given place, it is reasonably certain that the true means and extremes of run-off are compassed between at least as wide limits. Precipitation and temperature are being observed in the United States at more than 5 000 stations. Stream measurements are being made by Federal and State authorities and private parties, together, at about one-fourth as many stations.

Notwithstanding the valuable work being performed by these organizations on all too meager appropriations, relatively few stream-flow data are available. For most streams, only short-term records have been secured, covering by no means the extremes of high and low flow, or giving a dependable mean flow. If such measurements of stream flow as are available can be supplemented by reasonably accurate computed values, so as to give a long-term record of fair reliability, and covering more nearly the extremes of high and low flow, some of the uncertainty often attending efforts toward industrial utilization of the flow of streams and protection against floods may be eliminated.

It is the writer's hope that his method of computing run-off may at least be of assistance toward this end.

Before presenting the method itself, and its application to a number of widely different water-sheds, some of the basic data will be discussed.

Definition of Terms.

Rainfall.—This term, as here used, includes "snowfall" reduced to its equivalent in rain by melting the snow or by dividing by ten, according to the accepted practice of the United States Weather Bureau. Wherever precipitation occurs as light snow at temperatures well below the freezing point, this ratio gives values about 10 to 30%

high. However, inasmuch as at most observation stations a comparatively small portion of the precipitation occurs under the foregoing conditions, the resulting discrepancy in the annual precipitation is undoubtedly of less importance than other discrepancies, such as those resulting from differences in the placing of the rain gauge with respect to altitude and exposure. It is believed that, on the whole, however, precipitation measured with standard gauges, in accordance with Government practice, gives a quantity reasonably close to the truth.

Even though there may be some discrepancy in records of rainfall, giving quantities continually too large or too small, nevertheless, the computed run-off based on these data and on a comparison with available run-off measurements may be more accurate than the precipitation records on which the computations are based, since the constant error in the rainfall measurements becomes a part of the coefficient used in computing losses out of rainfall.

Evaporation.—This term is used to denote both the process of vaporization and the quantity of water which is vaporized and diffused into the atmosphere from land and water surfaces. Other writers have frequently included in the term "evaporation" both transpiration and deep seepage, in other words, all losses out of rainfall except run-off.

Transpiration.—This term is used to denote the water which escapes as vapor from the stomata of leaves, and the process by which such loss of moisture takes place. "Hygroscopic water", that is, the water retained in the vegetable substance produced, is inconsequential in this discussion.

Deep Seepage.—This is the water which passes down so far through the subsoil into the underlying rocks as to be lost to the given water-shed; it is also neglected on the streams discussed, although the possibility of its constituting an appreciable factor on one or two of the water-sheds considered, may here be noted.

Total Loss.—This term is used to denote the sum of evaporation and transpiration; and, whenever a long period of years is considered, the quantity "Precipitation minus Total Loss" is virtually equivalent to "Run-off". The monthly values of "Precipitation minus Total Loss", and to some extent also the annual values, however—depending on the size of the water-shed, and all the other factors which influence

the distribution of run-off—must be corrected for changes in ground and surface storage to give the run-off for the given time interval.

Run-Off.—This term is used to denote stream flow maintained by ground and lake storage, together with surface flow resulting from heavy rains or the melting of snow.

THE "WATER YEAR".

Frequent comment has been made on the fact that the calendar year is an inappropriate and conventional period into which to divide time, from a hydrological viewpoint. A period of 12 months, beginning December 1st and ending the following November 30th, has been used by many hydraulicians and called the "water year". This "water year" has again been divided into three periods, viz., December to May, inclusive, constituting the "storage" period; June to August, inclusive, constituting the "growing" period; and September to November, inclusive, constituting the "replenishing" period. Although this division of time is more logical than the calendar year, efforts to express run-off as a percentage of rainfall for each of these periods are considered by the writer hardly less futile than efforts to express run-off as a percentage of the monthly or annual rainfall.

Fig. 2* substantiates this view. The scale used in Rafter's diagrams for the growing and replenishing periods completely conceals the true lack of relationship between rainfall and run-off during these periods. At first glance one would conclude that the run-off during the growing and replenishing periods showed a much closer relationship to the rainfall than that of the storage period. On plotting the values for these two periods to a scale which results in a curve comparable to that of the storage period, however, quite the contrary is found to be the case, as shown in Fig. 3.

During the storage period, the run-off varies from 12.8 to 22.3 in., or practically 75% for a rainfall of between 22 and 23 in. During the growing period, on the same stream, the run-off varies from 0.72 to 3.07 in., or 325% for approximately the same rainfall. During the replenishing period the run-off varies from 3.76 to 1.58 in., or 140% for rainfalls of 13.11 in. and 12.89 in., respectively. The entire annual run-off from this water-shed varied in 15 years from 12.69 in.

* Compiled from "The Relation of Rainfall to Run-Off", by the late George W. Rafter, M. Am. Soc. C. E.

for 39.70 in. of rainfall to 23.27 in. for 38.71 in. of rainfall, or practically 100% for the same rainfall.

The writer usually takes as his rainfall year, in northern latitudes, the 12-month period beginning November 1st, and as the corresponding run-off year the 12-month period beginning the following March 1st. Stream flow during the winter, in the northern half of Minnesota, for example, is almost entirely independent of the precipitation during these months, because such precipitation is practically all stored as snow. Stream flow, in such latitudes, is dependent on the ground-water stored during the previous open seasons.

In the greater portion of the United States, a 12-month period beginning August, September, or October 1st, when the ground and surface storage are both reduced to a minimum, affords a satisfactory "water year". Usually, however, the annual yield of a water-shed, even in such "water years", is modified somewhat by ground storage.

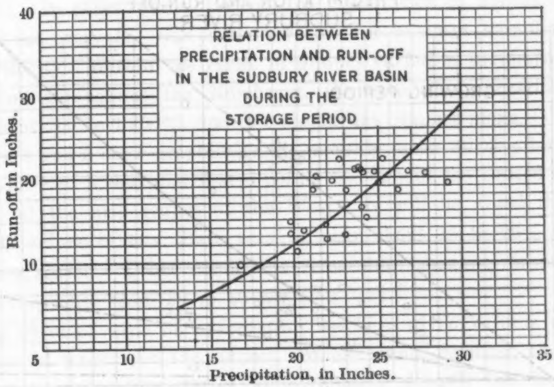
The writer computes the annual run-off entirely by calendar months, without any attempt to adhere to a division of the year into "storage", "growing", and "replenishing" periods, or into spring, summer, fall, and winter seasons.

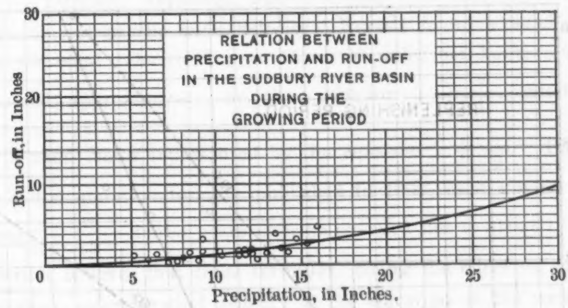
Evaporation from Water, Snow, and Ice.

Although the laws of evaporation, as stated by Professor Thomas Tate, are thirteen in number, the writer has found that in computing stream flow only three factors tending to modify evaporation need usually be considered. These are, in the order of their importance: temperature, humidity, and wind. The last two are relatively unimportant within the ordinary range of variation, but some allowance from water-shed to water-shed, and month to month, must be made for wide differences.

The writer has accepted the principle, first enunciated by Dalton, that the rate of evaporation from a water surface, other conditions remaining constant, varies almost, if not quite, as the difference between the maximum vapor pressure corresponding to the temperature of the water, and the pressure of vapor present in the atmosphere.

Inasmuch as maximum vapor pressure is a function of the temperature—doubling for every increase of approximately 18° Fahr.—the actual pressure of vapor present in the atmosphere also becomes a function of the temperature when the relative humidity remains


RELATION BETWEEN
PRECIPITATION AND RUN-OFF
IN THE SUDBURY RIVER BASIN
DURING THE
STORAGE PERIOD
Run-off, in Inches.
Precipitation, in Inches.
40
30
20
10
5
10
15
20
25
30
35


RELATION BETWEEN
PRECIPITATION AND RUN-OFF
IN THE SUDBURY RIVER BASIN
DURING THE
GROWING PERIOD
Run-off, in Inches
Precipitation, in Inches.
30
20
10
0
5
10
15
20
25
30

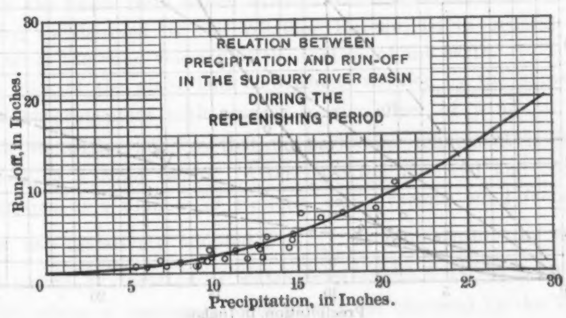

RELATION BETWEEN
PRECIPITATION AND RUN-OFF
IN THE SUDBURY RIVER BASIN
DURING THE
REPLENISHING PERIOD
Run-off, in Inches.
Precipitation, in Inches.
30
20
10
0
5
10
15
20
25
30

FIG. 2.

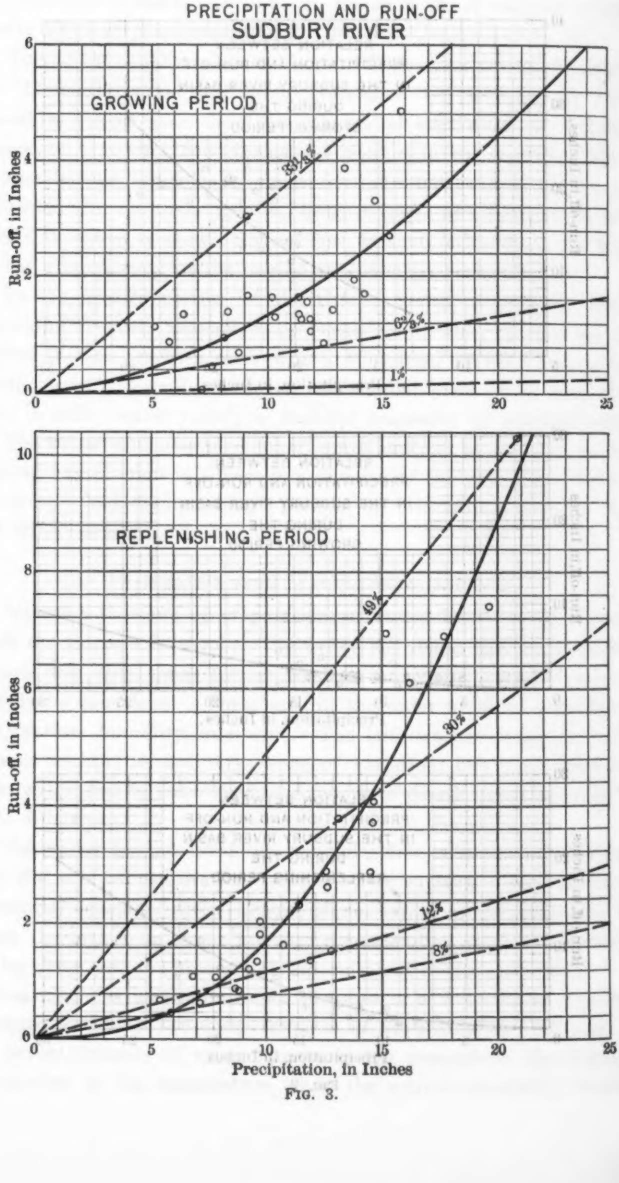

PRECIPITATION AND RUN-OFF
SUDBURY RIVER
GROWING PERIOD
33⅓%
6⅔%
1%
Run-off, in Inches
REPLENISHING PERIOD
49%
30%
12%
8%
Run-off, in Inches
Precipitation, in Inches

FIG. 3.

constant. In other words, the rate of evaporation is approximately doubled for every 18° rise in temperature for constant humidity and wind velocity. Within the range of annual variation in temperature prevailing throughout the Northwest, the rate of evaporation will vary about 700 to 1 200%, due to temperature changes alone.

Figs. 4 and 5 show graphically the monthly mean relative humidity at a number of widely distributed points throughout the United States, outside of the arid region of the West. It will be noted that the monthly values in any State vary by only about 15 to 20% during the open season. Other conditions remaining constant, evaporation would vary only from 30 to 50 per cent.

The effect of wind on evaporation has been given various weights by different writers on the subject. Wielenmann, Stelling, and Tate, hold that evaporation varies approximately directly as the wind velocity. DeHeen, Shierbeck, and Svenson, hold that it varies as the square root of the wind velocity. Russell found a wind factor which, for wind velocities up to 15 or 20 miles per hour, could be represented approximately by the equation $1 + \frac{w}{4}$. FitzGerald found a wind factor represented by the equation $1 + \frac{w}{2}$. Bigelow, in his first evaporation formula, published in 1908, used a wind factor of about $1 + \frac{w}{35}$. Three years later he changed this to about $1 + \frac{w}{11}$. The writer believes that these formulas, giving the effect of wind on evaporation, refer to wind velocity at the elevation of the water surface in the basin from which evaporation was measured. The wind velocity, as observed by the Weather Bureau, is, in general, about three times the wind velocity at the surface of the earth.

In view of the discordant conclusions reached by the above mentioned experimenters with respect to the effect of wind on evaporation, the writer believes that no particular refinement is justified in selecting a wind factor. After checking observed and computed evaporation at a number of stations, using various wind factors, the writer has tentatively adopted $1 + \frac{w}{10}$ as representing the effect of wind on the evaporation of water from surfaces at the level of the ground, where w represents wind velocity as observed by the Weather Bureau about 30 ft. above the general level of the surrounding country.

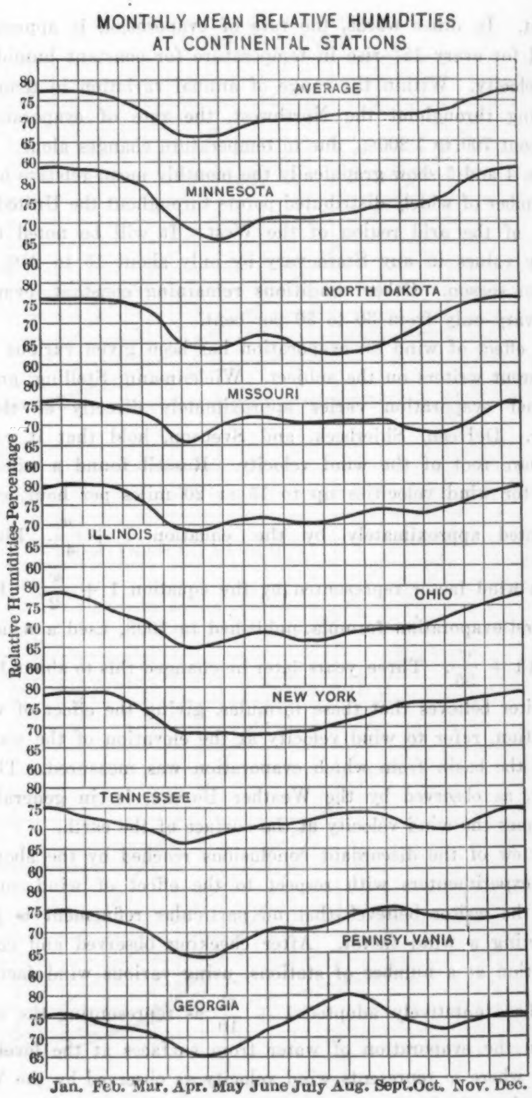

MONTHLY MEAN RELATIVE HUMIDITIES
AT CONTINENTAL STATIONS
AVERAGE
MINNESOTA
NORTH DAKOTA
MISSOURI
ILLINOIS
OHIO
NEW YORK
TENNESSEE
PENNSYLVANIA
GEORGIA
Relative Humidities-Percentage
80
75
70
65
60
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.

FIG. 4.

Using the writer's wind factor of $1 + \frac{w}{10}$, the annual variation in evaporation due to changes in monthly mean wind velocity amounts to only about 20 to 30 per cent. The monthly mean wind velocity at a number of stations in the eastern two-thirds of the United States is shown graphically in Fig. 6.

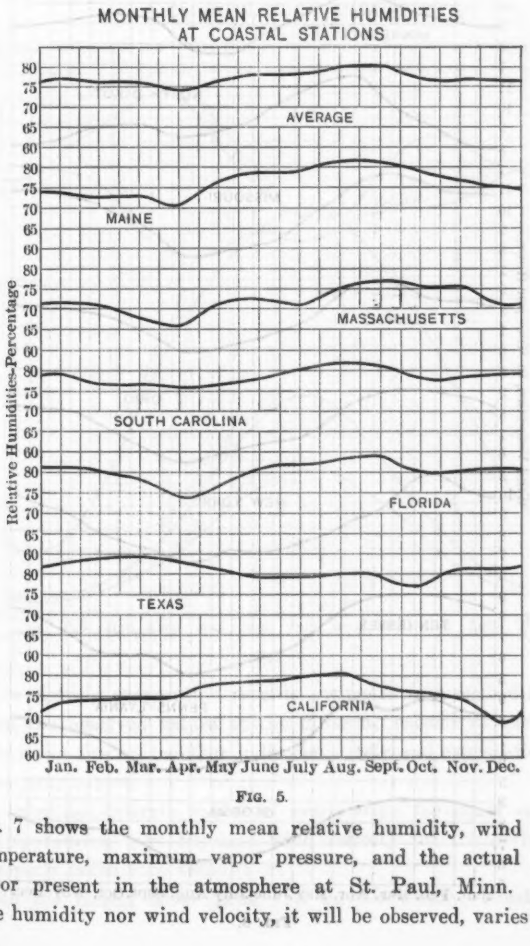


FIG. 5.

Fig. 7 shows the monthly mean relative humidity, wind velocity, air temperature, maximum vapor pressure, and the actual pressure of vapor present in the atmosphere at St. Paul, Minn. Neither relative humidity nor wind velocity, it will be observed, varies between

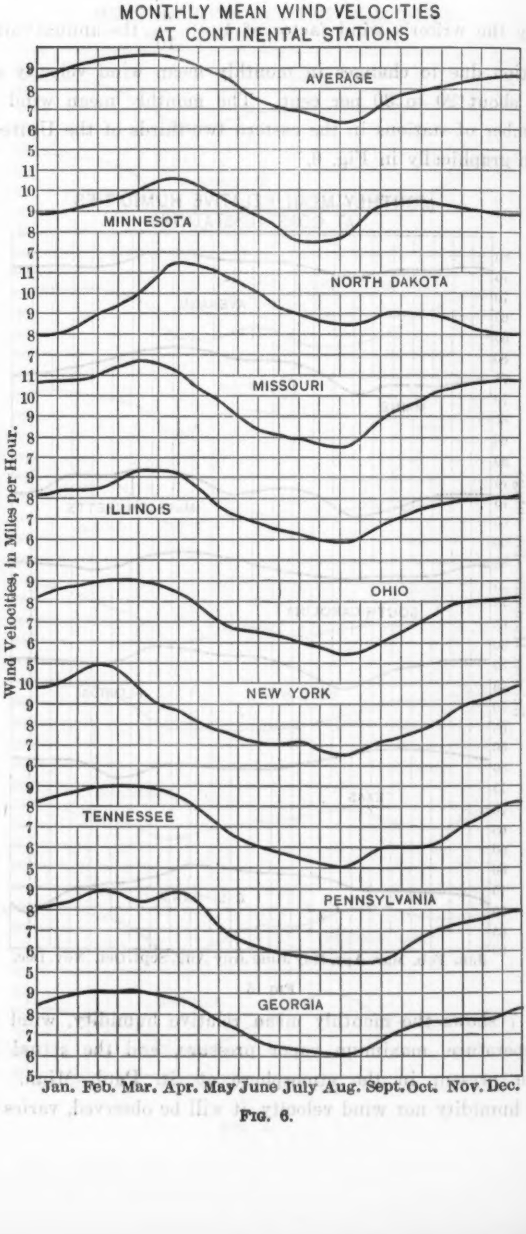

MONTHLY MEAN WIND VELOCITIES
AT CONTINENTAL STATIONS
Wind Velocities, in Miles per Hour.
AVERAGE
MINNESOTA
NORTH DAKOTA
MISSOURI
ILLINOIS
OHIO
NEW YORK
TENNESSEE
PENNSYLVANIA
GEORGIA
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.

FIG. 6.

wide limits. The vapor pressures and their differences, that is, the factor which virtually determines evaporation, all show variations of several hundred per cent.

Fig. 8 shows the evaporation from shallow water for different temperatures and seasons of the year, under conditions prevailing throughout the Northwest. A fundamental error has sometimes been

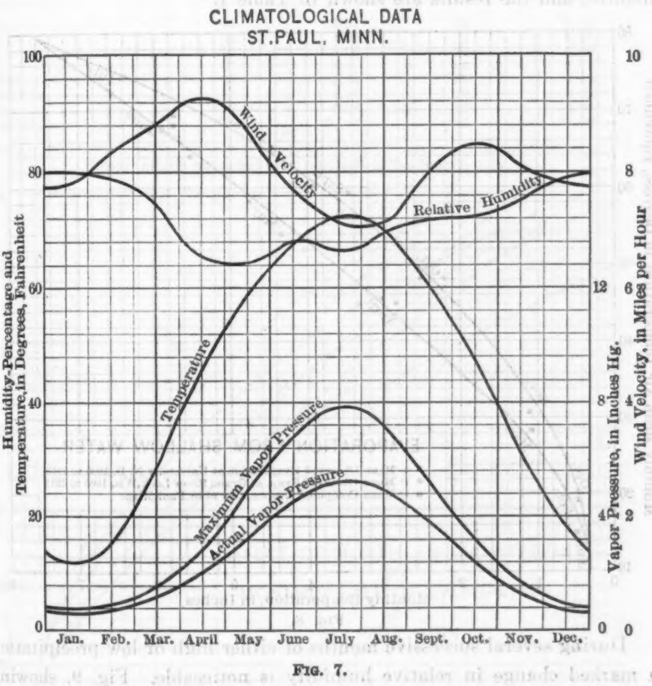


FIG. 7.

made in plotting evaporation records, owing to failure to differentiate between the observed values at the different seasons of the year, all months being thrown together indiscriminately and evaporation plotted against temperature.

By plotting separate values of evaporation for equal spring and fall temperatures, humidity and wind velocity for the given locality are properly taken into account.

Both the Grand River Lock and the University, North Dakota,

evaporation stations are on small shallow bodies of water. The mean relative humidity at the North Dakota station is slightly less, and the wind velocity slightly greater than at the Wisconsin station. Monthly observations have been grouped together and the average taken. Observations of temperature and evaporation extend over the years, 1905 to 1913, inclusive. These values have been averaged by months, and the results are shown in Table 1.

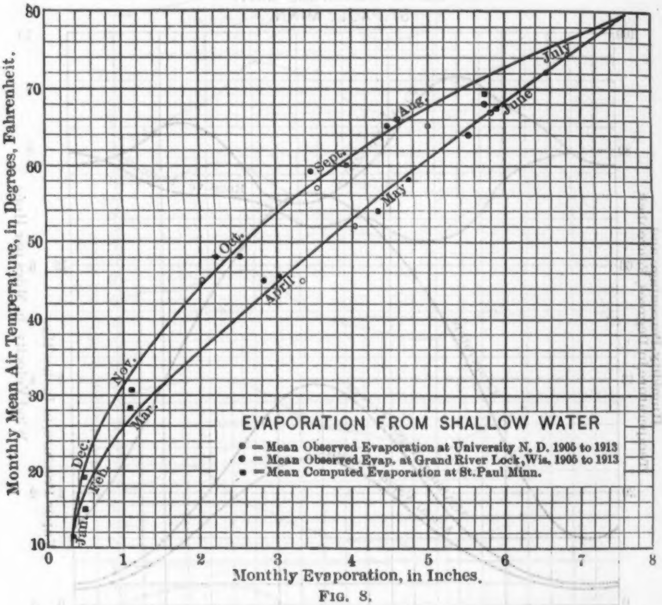


FIG. 8.

During several successive months of either high or low precipitation a marked change in relative humidity is noticeable. Fig. 9, showing the effect of high and low rates of monthly precipitation on evaporation, is computed from meteorological data for St. Paul, Minn. The effect shown must be considered more or less approximate, because based on relatively few data, yet indicative of the variation in monthly evaporation resulting from large variations in monthly precipitation, and hence basic to the later discussion pertaining to the proportion of precipitation lost by evaporation from land areas, at any given temperature, for various rates of rainfall. Both higher and lower

TABLE 1.—OBSERVATIONS OF TEMPERATURE AND EVAPORATION.

Month.	At University, North Dakota.		At Grand River Lock, Wisconsin.	
	Air temperature, in degrees Fahrenheit.	Evaporation, in inches per month.	Air temperature, in degrees Fahrenheit.	Evaporation, in inches per month.
April	45	3.37	45	2.83
May	52	4.03	54	4.35
June	65	5.00	64	5.52
July	67	5.81	68	5.74
August	66	4.60	65	4.46
September	57	3.57	59	3.45
October	45	2.02	48	2.22
November	*34	*0.52	*41	*1.09

*Observations extended over portions of the month only.

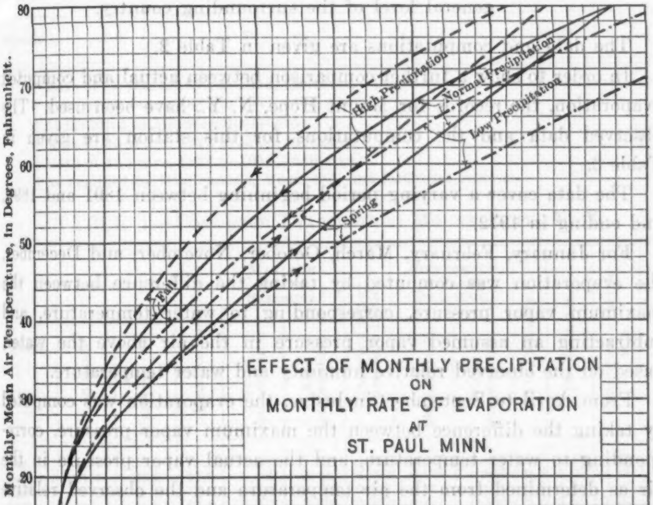


FIG. 9.

than normal rates of precipitation, in inches per month, seem to affect the rate of evaporation less in early spring than in fall. This might be expected because the moisture available for evaporation in the spring is less dependent on rainfall than it is in the fall, hence the

relative humidity would not be affected by changes in precipitation as readily in spring as in fall.

The evaporation at St. Paul, Minn., was computed by the following formula:

$$E = 15\,(V - v)\left(1 + \frac{w}{10}\right)$$

in which E = Evaporation, in inches per month;

V = Maximum vapor pressure, in inches of mercury, at monthly mean air temperature;

v = Actual vapor pressure, at monthly mean air temperature and relative humidity;

w = Wind velocity, in miles per hour, as measured by the Weather Bureau, approximately 30 ft. above the general level of the surrounding country.

The data and computations are given in Table 2.

In order to show a further comparison between actual and computed evaporation, the records for Mount Hope, N. Y., have been used. The observed data and the computations for this station are given in Table 3.

The data cover a varying period beginning between 1891 and 1896, and ending in 1912.

For January, February, March, October, November, and December, the evaporation was computed by taking the difference between the maximum vapor pressure, corresponding to water temperature, and subtracting an assumed vapor pressure in the air above the water, based on the observed relative humidity and water temperature.

From April to September, inclusive, the evaporation was computed by taking the difference between the maximum vapor pressure, corresponding to water temperature, and the actual vapor pressure in the air as determined from the air temperature and the observed relative humidity.

During the winter months the water in the floating tub was at a surprisingly high temperature, presumably due to the pumping of water into the reservoir in which the tub was floating. It is assumed, therefore, that the layer of air immediately above the water surface was heated to nearly water temperature, and that the most probable value for actual vapor pressure would be secured by applying the relative humidity to the maximum vapor pressure at the water tem-

TABLE 2.—Computed Evaporation at St. Paul, Minn.

Month.	Temperature.*	Maximum vapor pressure in Hg.	Relative Humidity. Mean of 8 A. M. and 8 P. M.†	Relative Humidity. Mean.‡	Difference in vapor pressure.	Wind velocity, in miles per hour.§	Precipitation.‖	Computed evaporation, in inches.
January...	11.6	0.069	80%	81	0.013	7.8	0.90	0.35
February..	15.0	0.081	79	79	0.017	8.3	0.84	0.47
March.....	28.2	0.151	74	75	0.038	8.8	1.60	1.07
April......	45.7	0.306	65	66	0.104	9.3	2.33	3.02
May........	58.2	0.485	64	65	0.170	8.7	3.12	4.76
June.......	67.4	0.670	68	67	0.221	7.7	4.41	5.88
July.......	72.1	0.786	66	¶67.5	0.255	7.1	3.40	6.55
August....	69.5	0.720	69	69	0.223	7.1	3.46	5.72
September	60.3	0.523	72	72	0.146	8.0	3.42	3.94
October....	48.1	0.335	72	73	0.091	8.5	2.34	2.53
November.	30.9	0.171	75	76	0.041	8.1	1.30	1.11
December.	19.3	0.099	79	81	0.019	7.8	1.06	0.51
Annual....	43.9	....	72	....	....	....	28.68	....

* Mean of 43 years, 1871 to 1913, inclusive.

† Mean of 24 years, 1888 to 1911, inclusive.

‡ As modified by Minneapolis records.

§ Mean of 36 years, 1873 to 1908, inclusive.

‖ Mean of 43 years, 1871 to 1913, inclusive.

¶ As modified by Minneapolis, Moorhead, and LaCrosse records.

TABLE 3.—Computed Evaporation, Mount Hope, N. Y.

Month.	Temperature. Air, in shade.	Temperature. Water, in tub.	Relative humidity.	Maximum Vapor Pressure. Water temperature.	Maximum Vapor Pressure. Air temperature.	Observed vapor pressure in air.	Difference in Vapor Pressure. Air temperature.	Difference in Vapor Pressure. Water temperature.	Wind velocity.*	Precipitation.	Computed evaporation.	Actual evaporation.
Jan.....	24.8	32.5	78.0	0.183	0.129	0.101	0.028	0.082	10.0	1.83	1.21	1.27
Feb.....	23.3	32.3	77.9	0.182	0.120	0.092	0.028	0.090	10.9	1.49	1.26	1.26
Mar	34.1	35.8	73.9	0.209	0.196	0.145	0.051	0.064	9.8	1.96	1.63	2.35
Apr	46.7	46.6	68.8	0.317	0.318	0.217	0.101	0.100	8.8	2.21	2.82	2.97
May	60.1	59.0	68.1	0.499	0.519	0.354	0.165	0.145	7.8	2.69	3.88	3.64
June....	70.5	68.2	68.1	0.689	0.744	0.507	0.237	0.182	7.2	2.47	4.70	4.40
July	74.7	72.6	68.8	0.799	0.858	0.590	0.268	0.209	7.2	3.70	5.40	5.11
Aug	72.1	71.3	71.3	0.765	0.786	0.560	0.226	0.205	6.5	2.91	5.08	4.73
Sept....	65.7	65.6	74.6	0.629	0.631	0.471	0.160	0.158	7.2	2.44	4.08	3.63
Oct.....	53.1	53.8	75.2	0.414	0.404	0.304	0.100	0.110	7.7	2.37	2.73	2.65
Nov	39.9	42.5	76.6	0.271	0.246	0.188	0.058	0.083	8.9	1.97	1.80	1.70
Dec.....	28.9	34.3	77.9	0.197	0.156	0.122	0.034	0.075	9.5	1.74	1.28	1.56
Annual.	49.5	51.2	73.2	....	....	......	....	....	....	27.77	......	....

* Wind velocity according to U. S. Weather Bureau at Rochester, N. Y., 1895 to 1906.

perature rather than at the air temperature observed some distance above the water surface. It is a fact, illustrated in Fig. 10,* that the average relative humidity varies only very slightly with changes of altitude of several thousand feet. This might be expected, because there is constant diffusion of vapor tending toward equal relative humidities. Tate says that vapor diffuses through the atmosphere much more rapidly than it is formed from the surface of the water,

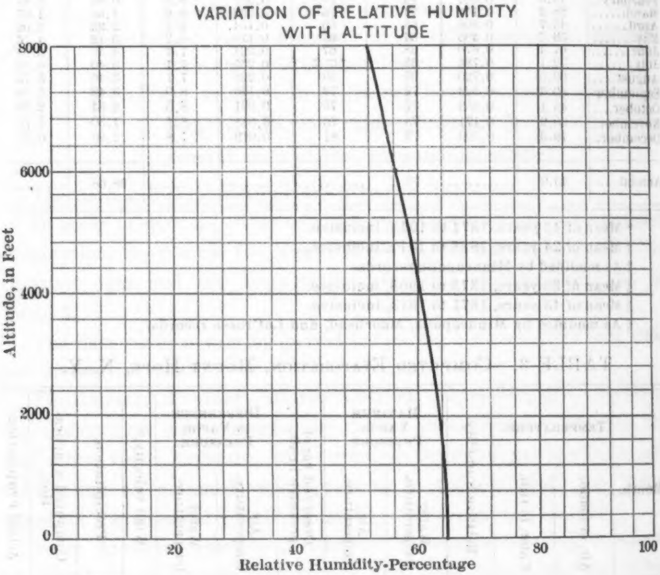


FIG. 10.

assuming no convection currents present. This rapid diffusion of vapor, even in spite of air currents, must tend strongly toward the maintenance of equal relative humidities at all altitudes.

Table 4 gives the monthly evaporation† as observed and estimated by Desmond FitzGerald, Past-President, Am. Soc. C. E., at Boston, Mass.

The values in Table 4 are platted in Fig. 11. The similarity between this curve and that of Fig. 8 is apparent.

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* Data from *Bulletin F*, U. S. Weather Bureau, 1898.

† *Transactions*, Am. Soc. C. E., Vols. XV and XXVII.

TABLE 4.—EVAPORATION AT BOSTON, MASS.

Month.	*Temperature, in degrees, Fahrenheit.	Observed values, 1875-1885.	Observed values, 1875-1890.
January	27	0.90†	0.96†
February	28	1.20†	1.05†
March	34	1.80†	1.70†
April	44.5	3.10†	2.97†
May	58	4.61	4.46
June	67	5.86	5.54
July	71	6.28	5.98
August	69	5.49	5.50
September	62	4.09	4.12
October	52	2.95	3.16
November	40	1.63	2.25
December	30	1.20†	1.51
		39.11	39.20

* Values taken from curve.
† Values largely estimated.

Many other records of evaporation are available, but are omitted here because they do not give the evaporation from shallow water under conditions of wind and humidity prevailing throughout the Northwest.

The curves in Fig. 12 give the evaporation from open water in lakes of medium size and depth, and also the evaporation from snow and ice surfaces. The high rate of evaporation from snow under certain exceptional conditions has frequently been commented on. The values given are believed to be approximately right for ordinary conditions. The break in the up-going curve results from the fact that relatively large bodies of water remain frozen in spring until the monthly mean temperature has reached about 38 degrees. Until the break-up the evaporation is from snow and ice surfaces, and consequently larger, at the same temperature, than that from the relatively cold water immediately after the break-up. Most bodies of water of moderate size freeze up in the fall, when the monthly mean temperature reaches about 20 degrees.

Horton gives some observations on evaporation from snow* which check the values indicated by the curves of Fig. 12 very well. A loss of 0.25 in. was recorded during the 9 days from December 26th, 1913, to January 4th, 1914. The mean maximum temperature during this

* *Monthly Weather Review*, February, 1914.

period was 29.5°, corresponding to a monthly mean temperature of about 24 degrees. The recorded evaporation for 9 days corresponds to a monthly evaporation of 0.83 in. The curve of Fig. 12 gives 0.9 in. of evaporation per month at a temperature of 24° under base conditions of wind and humidity. The mean wind velocity during the period of observation was 7.3 miles per hour. The relative humidity

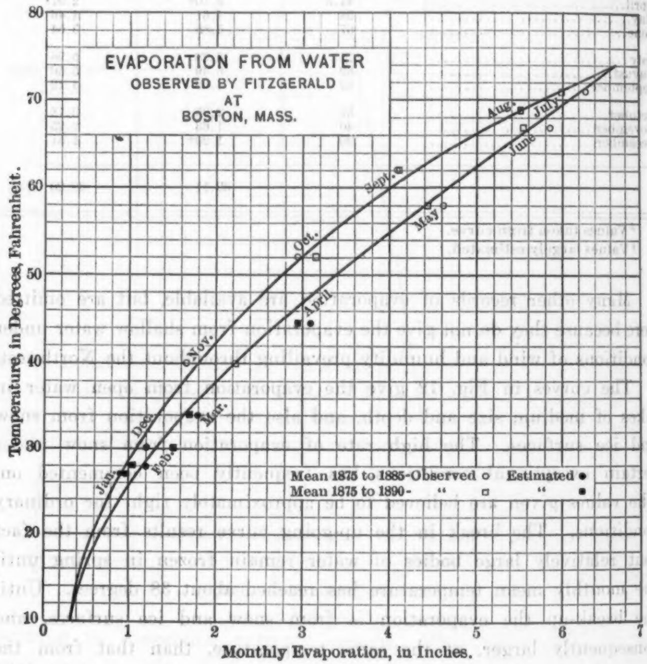

FIG. 11.

recorded by the Weather Bureau for January, 1914, was 78 per cent. These values represent approximately normal conditions for the Northwest.

Fig. 13 shows the monthly mean, and the mean maximum and minimum temperatures, and the approximate percentage of total precipitation which falls as snow during the various months of the year, at St. Paul and at Moorhead, Minn. For a monthly mean temperature of 23°, the mean of the maximum daily temperatures is

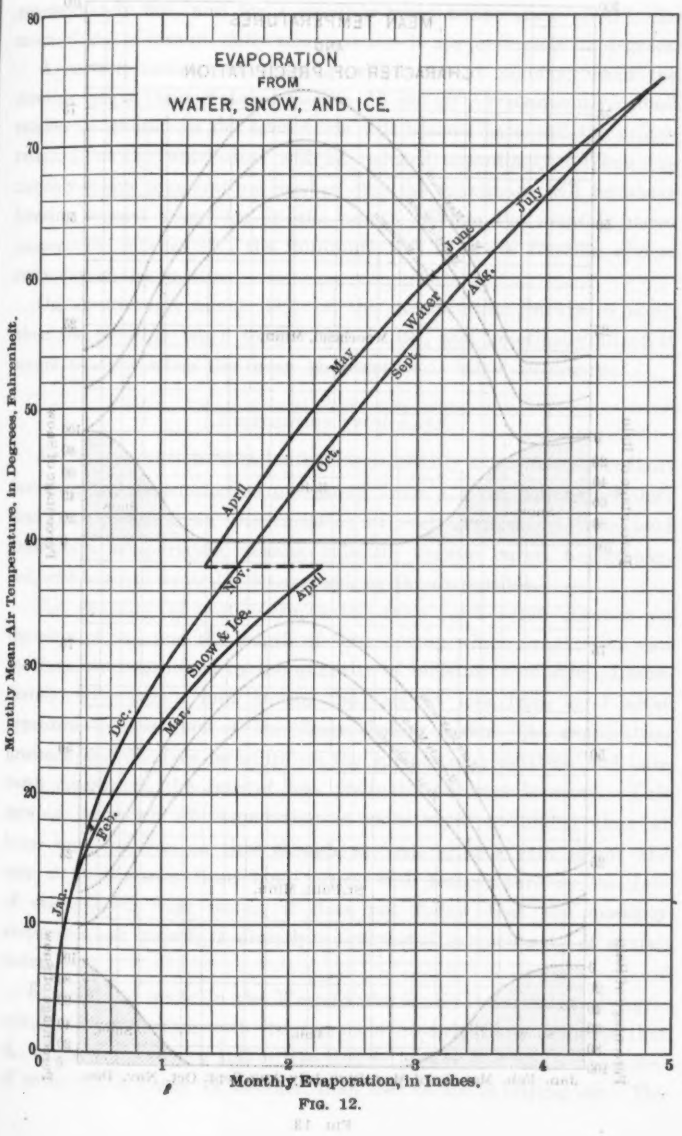
EVAPORATION
FROM
WATER, SNOW, AND ICE.
Monthly Mean Air Temperature, in Degrees, Fahrenheit.
Monthly Evaporation, in Inches.
0
10
20
30
40
50
60
70
80
1
2
3
4
5
Jan.
Feb.
Mar.
April
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.
Water
Snow & Ice.

FIG. 12.

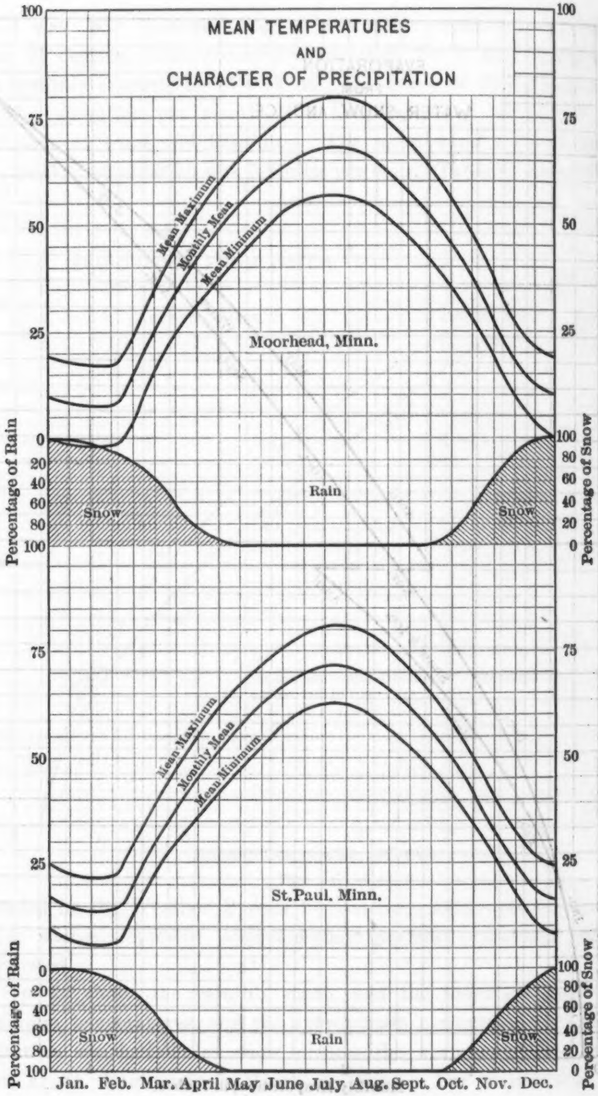
MEAN TEMPERATURES
AND
CHARACTER OF PRECIPITATION
100
75
50
25
0
Mean Maximum
Monthly Mean
Mean Minimum
Moorhead, Minn.
Percentage of Rain
0
20
40
60
80
100
Snow
Rain
Snow
Percentage of Snow
100
80
60
40
20
0
75
50
25
Mean Maximum
Monthly Mean
Mean Minimum
St. Paul, Minn.
Percentage of Rain
Snow
Rain
Snow
Percentage of Snow
Jan. Feb. Mar. April May June July Aug. Sept. Oct. Nov. Dec.

FIG. 13.

approximately 32°, and for a monthly mean temperature of 41°, the mean of the minimum daily temperatures is approximately 32 degrees. A certain amount of thawing will occur at midday, when the monthly mean temperature reaches 18 to 20°, consequently, small patches of ground in the open fields will become bare and the evaporation from the water-shed will be reduced accordingly. When the monthly mean temperature reaches 32°, the maximum will be above freezing almost every day in the month. When the monthly mean temperature reaches 48°, the minimum will be above freezing almost every day in the month.

On an average, about 30% of the precipitation occurs as snow when the monthly mean temperature is 40°, and practically all of it occurs as snow when the mean temperature is below 20 degrees.

Evaporation from Land.

As the evaporation from land areas is usually a far more important factor in the determination of run-off from a given water-shed than that from water areas, the variation of such evaporation from land areas with temperature, season, rainfall, vegetal cover, topography, soil, and subsoil, must be determined, as far as possible.

The quantity of water evaporated from land areas depends on the rates of loss and the length of time during which evaporation can continue, as determined by the quantity of moisture available. Immediately following a rain storm, the rate of loss from land areas approximates the rate of loss from shallow water. As evaporation proceeds and the free moisture on the surface of vegetation and bare earth disappears, the rate of loss gradually becomes lessened. This rate of reduction of evaporation is more rapid at higher than at lower temperatures, so that though at first, after a rain storm, the rate of evaporation from land varies with temperature as the rate of evaporation from water, it does not follow that the quantity evaporated per month is directly proportional to such rate of evaporation.

Experiments made in the West by the U. S. Department of Agriculture, in connection with its irrigation investigations, indicate that the temperature of dry soil in the sun is higher and the temperature of moist soil is lower, in summer, than that of the overlying air. The

assumption used herein, that the temperature of the moisture on the surface of the ground, on the surface of vegetation, and in the upper layers of the soil, is on an average, approximately equal to that of the air, is believed to be sufficiently close to the truth for present purposes. The monthly mean air temperature, then, becomes an index to the monthly mean rate of evaporation from land areas. It also virtually determines the quantity evaporated per month whenever there is sufficient precipitation to keep the ground saturated, or covered with snow.

Fig. 14 shows the relation between the moisture available in the soil and the quantity evaporated.* The soil experimented with was a well pulverized, sandy loam. The rapid decrease in the rate of evaporation after the percentage of moisture dropped below 10 is significant. When the moisture content of this particular soil dropped below 3.5% of its dry weight, evaporation practically ceased.

Fig. 15† shows the relation between "irrigation" and "evaporation". The irrigation water was applied twice a month, thus roughly approximating rainfall conditions. The constant of evaporation in the given case (the moisture content at the beginning of the experiment was 7.73%) was 0.15 in. For an irrigation of 1 in. per month, that is, for an approximate equivalent of 1 in. of rainfall per month, the evaporation was 0.15 in. plus 95% of the "rainfall". For an approximate equivalent of 2 in. of rainfall the evaporation was 0.15 in. plus 80% of the "rainfall". For 3 in. it was 0.15 in. plus 74% of the "rainfall". For 4 in. it was 0.15 in. plus 71% of the "rainfall". These facts again indicate that the evaporation loss varies much more rapidly at low than at the higher rates of rainfall. The recorded evaporation from a water surface averaged 10.9 in. during the period of the experiment from which the foregoing data were obtained.

It may here be remarked that, for an air temperature of 85°, corresponding to the air temperature for the experiment just quoted, the writer's curves of "Evaporation from Land Areas", shown later in Fig. 19, indicate a maximum evaporation of 7.3 in. from saturated soil under base conditions. Under Tulare, California, conditions,

* The data used in constructing this curve were taken from *Bulletin No. 177*, Office of Experiment Stations, U. S. Department of Agriculture.

† Also based on data secured from *Bulletin No. 177*, Office of Experiment Stations, U. S. Department of Agriculture.

agreeing with those of the foregoing experiment with respect to humidity, bare soil, and flat slope, a constant of 1.4 to 1.5 would be applied to the base values, giving from 10 to 11 in. as the maximum evaporation from a saturated soil surface under the given conditions. These base curves of evaporation, Fig. 19, indicate 15 in. of rainfall per month as permitting of maximum evaporation. The equation for the curve of evaporation, shown in Fig. 15, is 0.5 in. plus 62% of the "irrigation" (approximately "rainfall"). Applying this formula to 15 in. of rainfall gives 9.8 in. as the monthly evaporation.

Inasmuch as the application of irrigation water every 2 weeks only roughly approximates the condition of natural precipitation, the curve of Fig. 15 must be considered as merely indicative of how evaporation varies with rainfall.

The quantity of moisture available for evaporation, after a given fall of rain on any particular water-shed, depends mainly on the character and rate of precipitation, the quantity and kind of vegetal cover, the character of the soil, the slope of the ground, the temperature, and the season of the year. Torrential rains permit less evaporation than an equal quantity of precipitation in scattered showers. Snowfall permits relatively greater evaporation than rainfall.

The character of precipitation is well indicated by the temperature, and the hourly and daily rate of precipitation in any one season averages approximately the same from year to year in any locality, when a few exceptional storms are eliminated. It is realized, of course, that these exceptional storms are usually the ones which determine flood flow, particularly on small streams, even though they may not have material bearing on the annual run-off. Where the hourly and daily rates of rainfall are known to be radically different for two water-sheds, this factor can be taken into account in computing run-off, as will be explained later.

The character and quantity of vegetation on a given water-shed have a bearing on the loss out of rainfall from that water-shed, in that vegetal cover affects evaporation from land areas, and that the quantity of water used by plants varies with their character and growth. The quantity of rain and snow which never reaches the ground but is re-evaporated from the surfaces of vegetation also varies with the character and quantity of vegetation.

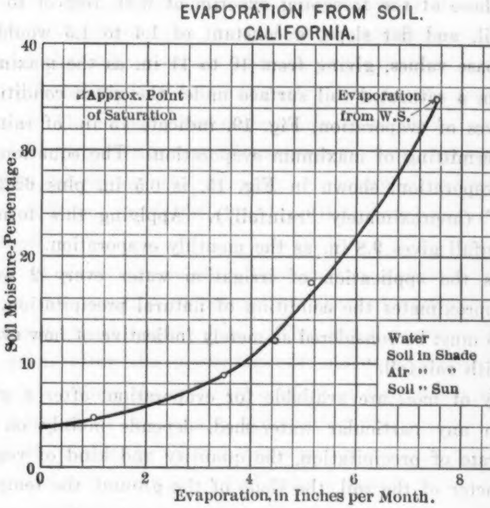
EVAPORATION FROM SOIL.
CALIFORNIA.
Approx. Point of Saturation
Evaporation from W.S.
Water 80°
Soil in Shade 78°
Air " 76°
Soil " Sun 108°
Soil Moisture-Percentage.
0
10
20
30
40
0
2
4
6
8
Evaporation, in Inches per Month.

FIG. 14.

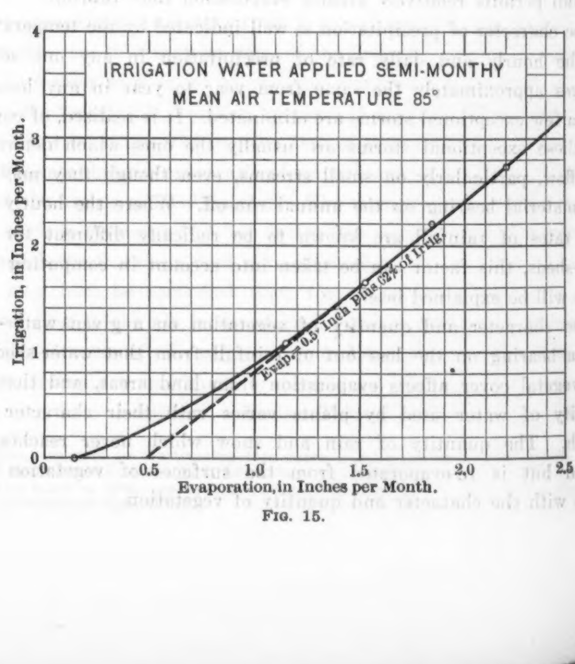
IRRIGATION WATER APPLIED SEMI-MONTHY
MEAN AIR TEMPERATURE 85°
Evap. 0.5" Inch Plus 62% of Irrig.
Irrigation, in Inches per Month.
0
1
2
3
4
0.5
1.0
1.5
2.0
2.5
Evaporation, in Inches per Month.

FIG. 15.

Unfortunately, experimenters in this field have usually stated the quantities intercepted by vegetation in terms of "percentage of precipitation". It is apparent, of course, that the quantity of precipitation intercepted by leaves, etc., depends largely on the quantity of rain or snow that falls in a given time, a much larger percentage being lost out of small showers than out of heavy rains. When moist snow falls on conifers, very large quantities are intercepted and more rapidly evaporated than if they had fallen to the ground in the open. Where water-sheds with a growth of only deciduous vegetation are under consideration, the distribution of rainfall throughout the year, particularly with reference to the growing season (although these are often coincident), also affects the quantity intercepted by such vegetation. In a general way, the interception by different forms of vegetation has been placed at from 12 to 30% of the rainfall.

It is a well-known fact that all forms of vegetation, particularly forests, shade the ground to a certain extent, and consequently reduce the rate of evaporation of free moisture. Whether or not they reduce the total quantity evaporated per month or per year depends also on the relative rates at which the rainfall can percolate into the forest floor, the cultivated field, and the bare ground, or run off into the streams.

Transeau* gives the following relative rates of evaporation observed at Cold Spring Harbor, Long Island:

Bare sand and gravel slide....................	100 per cent.
Open garden plot with low herbaceous vegetation..............................	80 to 100 " "
Upper beach areas............................	80 to 90 " "
Light forest on gravel soil..................	50 to 70 " "
Dense forest with abundant undergrowth.....	35 to 40 " "
Dense ravine forest with abundant herbaceous vegetation..............................	13 " "
Dense swamp forest with abundant undergrowth and water near surface.........	10 " "
Fresh-water marsh..........................	45 " "

The evaporation was measured with porous-cup atmometers placed about 4 in. above the surface of the ground.

* *Botanical Gazette*, April, 1908.

The loss of rainfall through evaporation (including reduced evaporation from the ground as counterbalanced by interception) from a dense deciduous forest has been assumed by the writer, for the present, as being about 15 or 20% less than the evaporation from an open grain field or from grass land. Evaporation from a dense coniferous forest has been assumed as greater than that from an open field during the winter and less in summer. Evaporation from a brush-covered water-shed has been assumed as somewhat less than that from open fields.

The quantity of water available for evaporation from any given water-shed is also influenced by the rate at which the rainfall can percolate into the ground or run off into the streams. Water-sheds with irregular, rough surfaces, and steep slopes, permit rapid surface run-off. Flat, or gently sloping water-sheds, of course, permit less rapid run-off, and hence greater evaporation. Sandy soils permit high rates of percolation. Clayey soils retard percolation, and facilitate surface run-off when on steep slopes, increase evaporation when on flat slopes, have great moisture-holding capacity, and exert a large capillary lift in bringing moisture to the surface for evaporation.

King* found the following rates of percolation through columns of sand and soil having a cross-section of 0.1 sq. ft. and 14 in. long, when kept covered with 2 in. of water:

In No. 40† sand, percolation at the rate of	301	in. per day.
In No. 100†	" " " " " "	39.7	" " "
In clay loam	" " " " "	1.6	" " "

Though it is very improbable that any but the most exceptional water-sheds of small area would have a surface covering of as coarse sand as No. 40, yet the differences between clayey and sandy soils, as to the facility with which rain water can percolate through them, is very marked.

The rate of percolation is also affected by the initial condition of the soil. When the upper layers become nearly air dry to any considerable depth, the pore space becomes so filled with air as to retard greatly the entrance of water. This is particularly true of the denser

* Nineteenth Annual Report, U. S. Geological Survey.

† No.	20	sand,	effective	diameter,	0.474 mm.
"	40	"	"	"	0.185 "
"	60	"	"	"	0.155 "
"	80	"	"	"	0.138 "
"	100	"	"	"	0.083 "

soils the individual pore spaces of which are relatively small, even though their moisture-holding capacity may far exceed that of the coarser sands. It has been found that ½ in. of precipitation will moisten the upper layers of soil sufficiently to establish contact with the lower layers containing more moisture, and thus increase, through capillarity, the moisture content of the upper layers of soil by a far greater quantity than the precipitation received. In this way ground-water may be brought to the surface and evaporation losses increased.

Another factor which affects the total quantity of evaporation from a given water-shed is the depth to which the percolating waters pass, and the ability of each particular soil to raise water to the surface again by capillary action. Experimenting on a series of cylinders, each having an area of cross-section of 0.1 sq. ft., and filled with a mixture of sand in approximately natural proportions, grains varying in size from No. 100 to No. 20, King found the following rates of evaporation for capillary lifts varying from 6 to 30 in. The temperature of the air in the laboratory where the experiment was conducted was about 70° Fahr., and the relative humidity is reported to have been very low.

Capillary lift, in inches above ground-water table.	Evaporation, in inches per month.
6	3.42
12	3.34
18	2.39
24	1.04
30	0.58

It is worthy of note that the maximum evaporation given in the above tabulation is only about one-half of what might be expected from a free water surface under the conditions of temperature and humidity stated. It is probable that if evaporation had been accelerated, capillarity would have shown itself equal to raising the moisture to the surface at a more rapid rate than that found by the experiment for capillary lifts of only a few inches.

The rates of evaporation given for a capillary lift of 30 in. indicate that when the water-table for the particular soil used in King's experiments drops to more than 4 or 5 ft. below the surface of the ground, evaporation is reduced to a very small quantity. This con-

clusion agrees with the statement of Charles H. Lee,* Assoc. M. Am. Soc. C. E., to the effect that capillary lift is practically limited to 4 ft. in coarse sandy soil, and to 8 ft. in fine sandy or clayey soil. It also agrees reasonably well with the conclusions of McGee,† stated in the following terms:

"While the effectiveness of capillary movement varies with the texture and structure of soil and subsoil and underlying rock, it may be said broadly that under average conditions capillarity acts freely to 4 or 5 feet in depth, fairly to 10 feet, and slowly to 30 or more feet."

Capillary action is facilitated by the rotted fibers of dead roots, which in some forms of vegetation penetrate to considerable depth.

In relation to ground-water, it is interesting to note that the "ground-water table" is not exactly what the word "table" might imply. On the basis of a detailed study of an area of about 1 200 by 1 800 ft., on the shore of Lake Mendota, Wisconsin, King found that the ground-water "table" presented undulations roughly conformable with the relief of the ground surface.

As the slope of the ground-water surface toward the stream affects the rate at which the ground-waters will be discharged into the stream, the rise in ground-water level after a given quantity of water has been allowed to percolate through soils of various moisture-holding capacities, has considerable bearing on the resulting increase in stream flow for any given quantity of percolation.

Fig. 16, based on King's experiments,‡ shows the quantity of water held by capillarity in coarse and fine sands at various heights above the water-table, after thorough drainage had been permitted for 2½ years. Evaporation from the surface was prevented. The greater moisture-holding capacity of fine sand is well illustrated by these curves.

In discussing the rise of ground-water level as the result of precipitation and percolation, King states that only about ¼ in. of water would be required to raise the ground-water level in the fine sand 1 ft., and that about ¾ in. would be required to raise that an equal distance in the coarse sand. This deduction relative to the great rise of ground-water level following a given percolation, and the difference in the rise in ground-water level in coarse and fine sands due to the same quantity of percolation, does not seem warranted. King appears to have reached his conclusion by computing the additional quantity

* *Transactions*, Am. Soc. C. E., Vol. LXXVIII, p. 148.
† 1911 Year Book of the Department of Agriculture, p. 482.
‡ Nineteenth Annual Report, U. S. Geological Survey, pp. 67 to 294.

of water required to produce saturation in the given sand up to a point 1 ft. above the water-table, assuming that no additional water would be lifted by capillarity above the original ground-water level. This reasoning is illustrated by the small triangular shaded area in Fig. 16. It appears to the writer, however, that the quantity of moisture required to raise the ground-water level 1 ft. would be indicated approximately by the lightly-shaded area between the broken and solid-line curves in Fig. 16, based on the assumption that the moisture held by capillarity is determined by the height above the ground-water table.

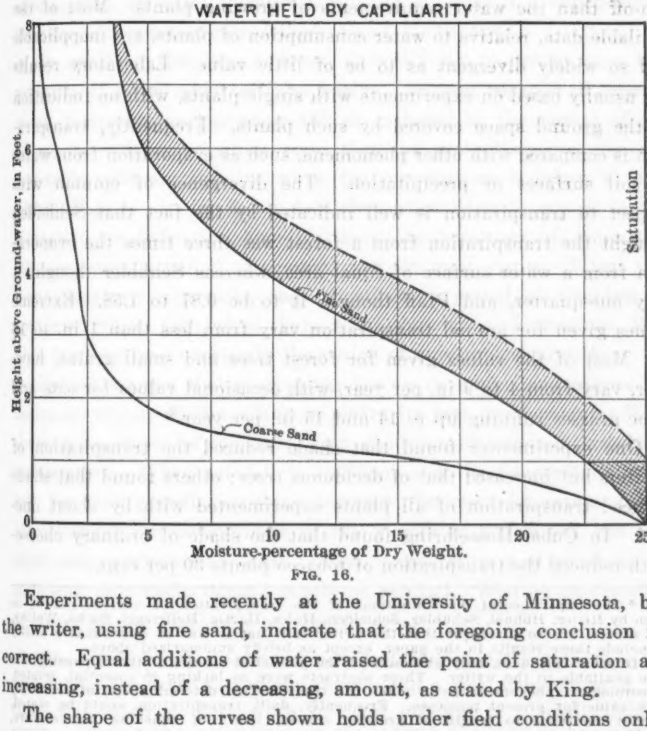

FIG. 16.

Experiments made recently at the University of Minnesota, by the writer, using fine sand, indicate that the foregoing conclusion is correct. Equal additions of water raised the point of saturation an increasing, instead of a decreasing, amount, as stated by King.

The shape of the curves shown holds under field conditions only when the point 8 ft. above the ground-water level is sufficiently far below the surface of the ground to be unaffected by surface evaporation. Under all other conditions the curves would be flattened.

It is probable that, under ordinary field conditions, 1 in. of rain soaking deeply into the ground will raise the ground-water level from 4 to 6 in., depending on the character and moisture content of the soil. A given addition to the ground-water will result in a greater increase in seepage flow to streams lying in clayey soils than in sandy soils, but the total ground-water supply to streams in sandy soils will be the greater because a larger percentage of rainfall is absorbed by such soils.

Transpiration.

Perhaps no more uncertain factor enters into the computation of run-off than the water requirements of growing plants. Most of the available data, relative to water consumption of plants, are inapplicable and so widely divergent as to be of little value. Laboratory results are usually based on experiments with single plants, with no indication of the ground space covered by such plants. Frequently, transpiration is compared with other phenomena, such as evaporation from water or soil surfaces or precipitation. The divergence of opinion with respect to transpiration is well indicated by the fact that Schleiden thought the transpiration from a forest was three times the evaporation from a water surface of equal area, whereas Schübler thought it only one-quarter, and Pfaff thought it to be 0.87 to 1.58. Extreme values given for annual transpiration vary from less than 1 in. to 16 ft. Most of the values given for forest trees and small grains, however, vary from 4 to 9 in. per year, with occasional values for oats and some grasses running up to 14 and 15 in. per year.*

One experimenter found that shade reduced the transpiration of conifers but increased that of deciduous trees; others found that shade reduced transpiration of all plants experimented with by about one-half. In Cuba, Hasselbring found that the shade of ordinary cheese-cloth reduced the transpiration of tobacco plants 30 per cent.

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* The experimental determinations of the transpiration of various plants, as given by Risler, Höhnel, Schübler, Schleiden, Hales, Hartig, Hellriegel, Sachs, Wollny, and others, are so divergent that the writer felt that it was of questionable utility to include these results in the paper, except as briefly summarized above.

In most instances, only abstracts of the published results of these investigators were available to the writer. These abstracts were so lacking in essential, related, meteorological phenomena as to make the transpiration determinations of relatively little value for present purposes. Frequently, daily transpiration would be stated, without reference to length of growing season, hours of sunshine, temperature, humidity, etc.

In published abstracts of the investigations above referred to, deductions, as to water consumption of plants, have frequently been made, based on some assumed length of season, and the like. Where experimental data were given for single trees, for example, a certain number of trees were assumed per acre, for the purpose of deducing a value of transpiration in inches depth on the ground area.

Transpiration, being the vaporization of water from the breathing pores or stomata of leaf and other vegetable surfaces, must necessarily depend largely on the same factors as evaporation. Most important among these is temperature. The law first stated by Van't Hoff and Arrhenius, that most chemical reactions and physiological processes double in activity for every rise in temperature of 10° cent., is quite firmly established. It has been found to be substantially correct for the rate of fixation of carbon dioxide by plants in sunlight; and, inasmuch as transpiration occurs during the process of carbon dioxide assimilation when the stomata open in the sunlight, it is reasonable to assume that the rate of transpiration, in so far as it is dependent on temperature, substantially follows Van't Hoff's law.

In applying this law, however, it is difficult to decide on a temperature at which plant activity begins. Köppen regards all monthly mean temperatures less than 48° as included in the period of rest of plants. Other scientists state that the protoplasmic contents of vegetable cells are inactive while the temperature is below 6° cent. (42.8° Fahr.). In temperate latitudes, when there is a lack of precipitation or irrigation, monthly mean temperatures of more than 72° constitute a period of summer rest for most plants. When sufficient moisture is present, they constitute a period of ripening for southern fruits, and, when there is an abundance of moisture, these high temperatures constitute the period of sub-tropical growth.

Botanists agree that every plant has its optimum temperature and moisture values, during which it makes its best development. When there is an excess of moisture, crop yields are determined largely by temperature. When rainfall is sufficient and the temperature is too low for best growth, sunshine becomes the most important factor. Heat cannot replace sunlight in the growth of vegetation, but sunlight can partly replace heat. When temperature and sunshine are sufficient, crop yields depend mostly on rainfall. Writing on the effect of water on the yield of corn, Smith says:*

"The grain plant obtains a great deal of its total weight from the soil during the early part of its growth, and a lack of moisture at this time will cause a short straw but not necessarily a small yield of grain. During the latter part of the growth the seed is being made chiefly from material stored in the stalk, and moisture must be present

* *Monthly Weather Review*, February, 1914.

to flush the material from the stalk into the head or the grain will be shrunken."

Fig. 17 shows the writer's "Base curve of Transpiration", founded largely on Van't Hoff's and Arrhenius' law. When the moisture supply is ample, mixed vegetation native to the Northwest will have a normal transpiration of about 10 in. per season. Taking off from the base curve, the transpiration, in inches per month, corresponding to the monthly temperatures prevailing in Central Minnesota, for example, and summing up these monthly values, gives a normal seasonal transpiration of practically 10 in. If, on the other hand, the monthly temperatures prevailing in Mississippi, for example, are used in determining monthly transpiration, the total seasonal transpiration—as determined by temperature alone, without consideration of the other factors which modify these values—in Mississippi would be about twice as much as in Minnesota, according to the base curve of Fig. 17. On some water-sheds where the moisture supply is always ample, monthly transpiration is dependent on temperature, sunshine, and the form of vegetation. On other water-sheds, such as those in Mississippi, the available supply of moisture is usually the governing factor, within proper temperature limits. The curve of Fig. 17, at best, can serve merely as one of the guides in arriving at the monthly distribution of a given quantity of seasonal transpiration. Unrestricted transpiration, within the limits of utilization of the given form of vegetation, occurs on few water-sheds even for all the months of the growing season in any year. A deficiency of rainfall in a given month, especially in spring, usually affects the transpiration for all the remaining months of that season, particularly in the case of trees and cultivated grains. Grasses respond more quickly to increased moisture supply. The monthly transpiration, then, will seldom be a constant for any water-shed from year to year, even for the same form of vegetation.*

* The application of Fig. 17 may be illustrated further, in this manner:

MISSISSIPPI RIVER WATER-SHED, 1901. TABLE 7.

Month.	Monthly Temperature.	Transpiration, off Curve.
April	41.7	0.2
May	57.1	1.5
June	62.1	1.9
July	69.9	2.4
August	65.2	1.9
September	55.1	1.0
October	46.7	0.3
Total		9.2

Similarly, the normal transpiration, as shown by the curve for the temperature prevailing on the same water-shed in 1907, gives 8.1 in. To indicate the great range

Most experimenters have found that the quantity of water transpired by plants varies approximately as the quantity of dry substance produced. Whether or not this relationship is purely accidental does not invalidate the fact. In the 1903 Year Book of the U. S. Department of Agriculture is given the average yield of corn for 15 years in the principal corn-growing States, together with the average precipitation over those States during June, July, and August. When platted, these data indicate an average yield of 5 + 2 bushels for every inch of rainfall during June, July, and August, between the limits of yields of 15 and 35 bushels per acre.

The data on water requirements of crops, recorded in *Bulletin 177,* Office of Experiment Stations, and *Bulletins 130, 188,* and *201,* Bureau of Plant Industry, U. S. Department of Agriculture, though not conclusive, indicate that the yield of grain is approximately proportional to the quantity of water consumed. The great difficulty encountered in the above mentioned experiments was that of differentiating between the evaporation from the soil and transpiration.

Livingston* gives considerable experimental data which show an almost direct relationship between transpiration and weight of vegetable substance produced.

In view of the substantially constant relationship for any given species of plant, which has been found by most experimenters between transpiration and vegetable substance produced, yields of hay, grain, etc., become a convenient index to the approximate relative quantities of transpiration to be expected on different water-sheds, and on the same water-shed in different years.

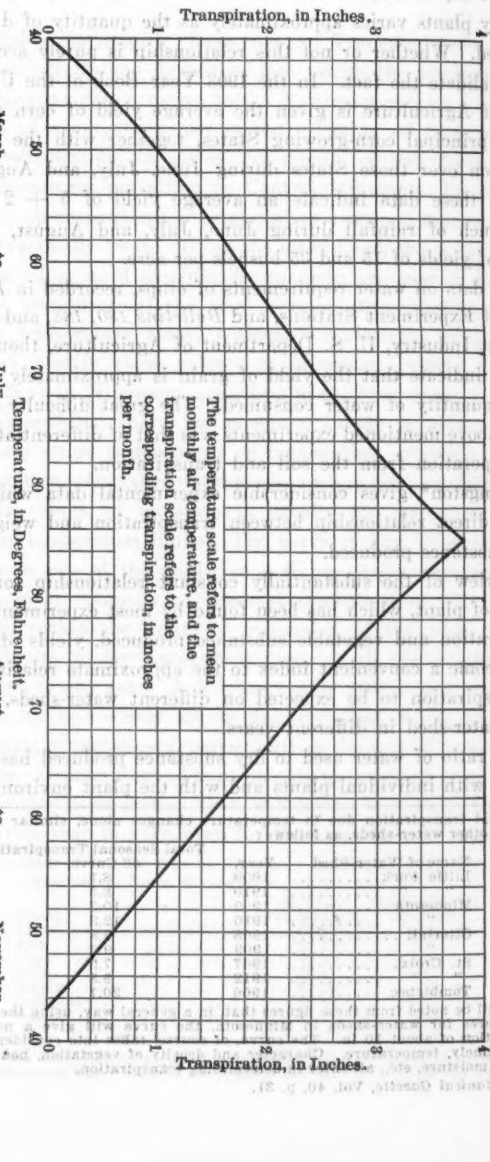
The ratio of water used to dry substance produced has been found to vary with individual plants and with the plant environment. Con-

of seasonal transpiration due to temperature changes alone, similar values may be given for other water-sheds, as follows:

Name of Water-Shed	Year.	Total Seasonal Transpiration, off Curve.
Little Fork	1909	8.1
" "	1910	8.5
Minnesota	1909	10.3
"	1910	12.3
Ottertail	1908	9.8
"	1909	9.3
St. Croix	1907	7.5
" "	1912	9.8
Tombigbee	1906	20.1

It will be noted from these figures that, in a general way, using the mean monthly temperatures for water-sheds in Minnesota, the curve will give a normal seasonal transpiration of about 10 in. The curve, of course, takes into consideration only one factor, namely, temperature. Character and density of vegetation, hours of sunshine, available moisture, etc., all enter in determining transpiration.

* *Botanical Gazette,* Vol. 40, p. 31.



BASE CURVE OF TRANSPIRATION

FIG. 17.

ifers, in particular, have been found to use less than deciduous trees; in fact, they are said to use less than one-sixth as much. For grass and grain, the ratio of pounds of water used to pounds of dry substance produced seems to vary from about 300:1 to 600:1. Some writers say deciduous trees use more water than grasses and grains, and others claim just the opposite. Something seems to depend on each particular writer's view as to the effect of forests on climate, stream flow, and kindred subjects.

The total annual transpiration for mixed vegetation, as previously stated, seems to vary between 4 and 9 in.

Most students of the subject of transpiration seem to be agreed that the quantity of water used by plants during the growing season depends mainly on the quantity available within reach of the root system. If plants transpired a quantity of water equal to from one-half to two times the evaporation from an equivalent surface of water, as claimed by some experimenters, a great many streams in the United States that have a very appreciable sustained flow would become intermittent, because there would be no ground-water supply to feed them. Surface run-off alone would appear in these streams.

It has been found that in any given soil all forms of vegetation wilt when the moisture content is reduced to a certain percentage. This percentage, however—known as the "wilting coefficient"—varies greatly for different soils. Fig. 18 shows the wilting coefficient and the moisture-holding capacity for various soils, expressed in percentages of the dry weight of the soil. The area between the two curves represents graphically the maximum quantity of moisture, in percentages of dry weight, which different soils can hold available for plant growth. The curves are based on data secured from *Bulletin 230,* Bureau of Plant Industry, U. S. Department of Agriculture.

In view of the fact that the moisture-holding capacity was determined from short soil columns, 1 cm. in length, it does not represent the moisture-holding capacity of long columns of similar soils under field conditions of drainage, yet gives an indication of such relative capacities. It may be noted that the moisture-holding capacity of sand agrees with similar values taken from the curves of Fig. 16, at the point of saturation.

Wilting coefficients and moisture-holding capacities, when expressed in percentages of dry weight, as is usually done, give a

somewhat distorted impression as to the relative capacities of sand and clay, because of differences in the dry weight of these materials. The actual maximum quantity of water available for plant growth varies from about 3.75 in. per ft. of depth of sand to 6.5 in. per ft. of depth of heavy clay, or about one-half the difference shown between these materials when available water is expressed in percentages of dry weight, as is usually done.

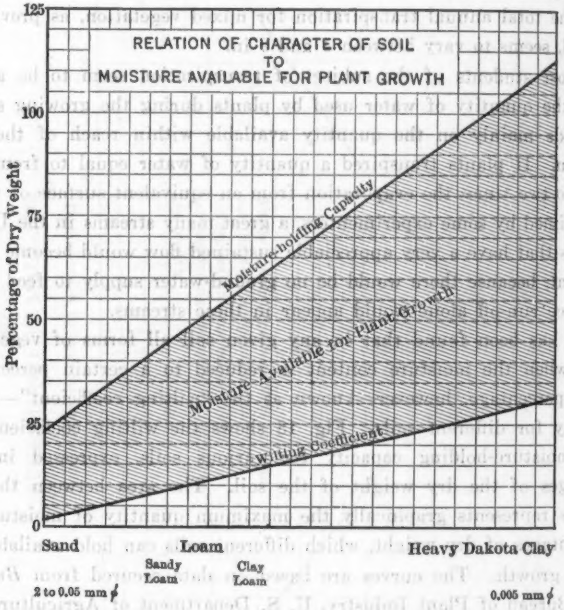


FIG. 18.

The true quantity of moisture available for plant growth under field conditions would seem to be given by the difference between the wilting coefficient and the moisture held in different soils by capillarity when free drainage is permitted, plus the quantity which can be absorbed by the plants during rains and before the excess moisture has had time to percolate into the subsoil out of reach of the root system. Expressed in inches of depth over the surface, the moisture available for plant growth after drainage under field con-

ditions, would appear to vary from about 1 to 2 in. per ft., depending on the character of the soil.

Even though all plants have been found to wilt when the moisture content for a given soil has been reduced to a certain percentage, the fact must not be lost sight of that in most fields the character of the soil varies greatly from foot to foot of depth, and that the roots of different forms of vegetation penetrate to widely varying depths, usually adapting themselves, in a measure, to the available moisture content of the various soil layers. Frequent light sprinkling of lawns is well known to make the grass non-drought-resistant, because it coaxes the root system to the surface where the greatest supply of moisture is temporarily found.

Transpiration of deep-rooted vegetation will be less fluctuating with varying monthly and annual rainfall than that of shallow-rooted vegetation. In dry seasons deep-rooted vegetation will draw more on ground storage. The transpiration of plants growing in sandy soils will vary more with rainfall than that of similar plants growing in clayey soils, other conditions remaining the same.

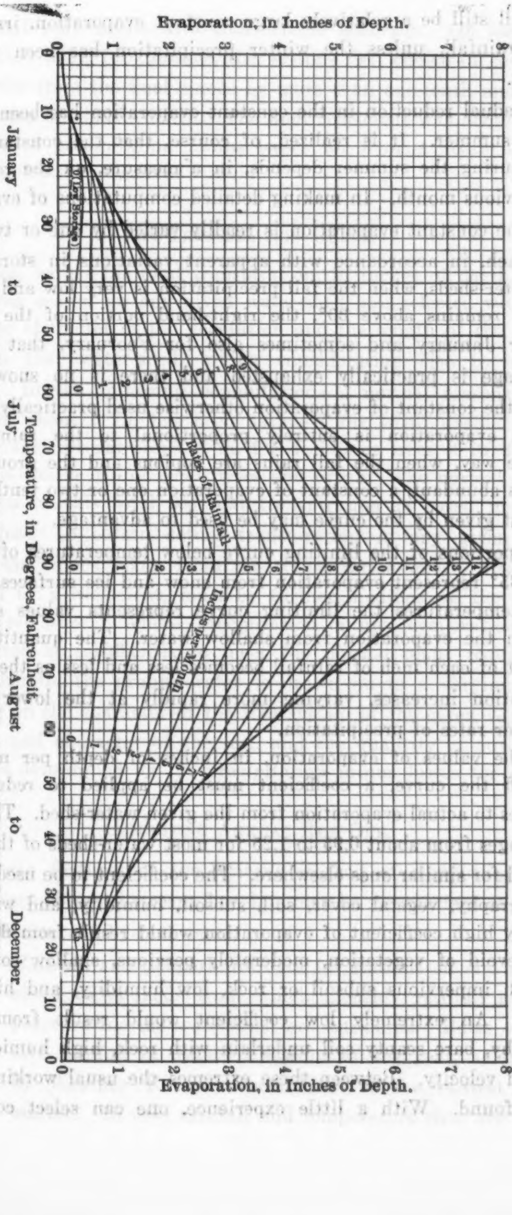
In estimating the water requirements of vegetation, the density of timber growth is not as important a factor as might at first appear. Cut-over water-sheds quickly grow up to grasses, weeds, and herbs of various kinds, which in turn are soon supplanted by shrubbery, brush, and then a growth of young trees. Areas of agricultural land not under cultivation, or after harvest and before fall plowing, soon become thoroughly covered with weeds and grasses, and hence suffer a transpiration loss perhaps fully as high as though seeded to grains and other useful vegetation. Even the rugged water-sheds of mountain ranges below the timber line are well covered with brush, grasses, moss, and other forms of low-growing vegetation. Burnt-over water-sheds with scanty covering of soil, and rock outcropping everywhere, as in the northeastern part of Minnesota, are also well covered with vegetation of one kind or another. As a consequence, the normal transpiration loss, so far as it is determined by the character of vegetation on different water-sheds, does not vary between wide limits. Except in the arid region, hardly a water-shed of considerable size can be found that is given over purely to one class of vegetation. Practically all of them are covered by mixed vegetation, including trees, shrubs, grasses, or grains. Of the streams considered, only

the three California rivers show a wide variation in transpiration losses. It is interesting to note, however, that this difference in transpiration loss does not result so much from a difference in the character of vegetation which could grow on the water-shed, so far as temperature is concerned, as a difference in the character of the vegetation which does grow on the given water-shed, as a result of differences in the quantity of water available for transpiration. For example, the grasses on the pastures of the Pit River Basin (see "Description of Water-sheds") were estimated to transpire 0.4 in. during April, 1907, when the monthly temperature was 48°, and 0.8 in. during May of the same year, when the monthly temperature was 52 degrees. The transpiration for the cultivated Root River water-shed, of Minnesota, in April, 1908, when the monthly temperature was 47° and the precipitation 2.9 in., was estimated at 0.3 in. The transpiration on the same water-shed for May, 1910, when the monthly temperature was 53° and the precipitation 2.6 in., was estimated at 0.9 in. On account of deficient precipitation, the transpiration on the Pit River water-shed during the summer, however, is limited to about 1 in., as compared with a summer transpiration on the Root River water-shed of more than 6 in.

The Evaporation Curve.

Although most of the facts on which the curve of evaporation from land areas, for various temperatures and rates of rainfall (Fig. 19), is based, have been discussed previously, the following more prominent considerations may bear further emphasis.

In the fall, when the monthly temperature reaches 20°, practically all the precipitation occurs as snow; consequently, evaporation for temperatures below 20° is no longer dependent on precipitation after the ground has been covered with snow, but entirely on temperature. Full evaporation, corresponding to the given monthly temperature, is usually possible throughout the winter. After the temperature rises above 20°, in spring, the evaporation again depends largely on available moisture, as determined mainly by precipitation. Nevertheless, a considerable constant evaporation is still possible, irrespective of precipitation, because a certain quantity of snow and ice is almost always present on the ground while the monthly temperature ranges from 20 to 35 degrees. After the snow has disappeared,



EVAPORATION FROM LAND AREAS FOR VARIOUS TEMPERATURES AND RATES OF RAINFALL.

FIG. 19.

there will still be a relatively large constant evaporation, irrespective of the rainfall, unless the winter precipitation has been distinctly deficient.

A gradual reduction in the constant evaporation has been assumed for the summer. It is realized, of course, that the constant evaporation during the summer depends, in a measure, on the rainfall of each previous month. In making detailed computations of evaporation losses, the constant evaporation is readily varied by one or two tenths of an inch, in accordance with apparent variations in storage. On some water-sheds, when the fall precipitation is very low and the temperature remains above 30°, the right-hand portion of the curve is used for January and sometimes also for February, that is, when the storage is practically exhausted and there is no snow on the ground, the constant of evaporation otherwise used practically vanishes and the evaporation is entirely proportional to the rainfall. In the same way, when the fall rains are copious and the ground-water supply is abundant, a constant of evaporation one or two tenths higher than that given by the curve may be used to advantage.

The portions of the limiting curve below temperatures of approximately 35° represent evaporation from snow and ice surfaces. At the higher temperatures the limiting curve represents values somewhat less than the evaporation from shallow water. The quantity evaporated out of each inch of rainfall becomes less and less as the monthly precipitation increases, varying more rapidly at the lower than at the higher rates of precipitation.

To the values of evaporation, in inches of depth per month, as taken off the curve, a coefficient must be applied to reduce these quantities to actual evaporation from the given water-shed. This coefficient ranges from about 0.95 to 1.25 for most water-sheds of the Northwest, and for similar ones elsewhere. The coefficient to be used depends on topography, vegetal cover, soil, subsoil, humidity, and wind. An extremely high coefficient of evaporation would result from flat topography devoid of vegetation, moderately pervious, shallow soil underlain with impervious subsoil or rock, low humidity, and high wind velocity. An extremely low coefficient would result from rugged topography, bare scanty soil underlain with rock, high humidity, and low wind velocity. Between these extremes the usual working values will be found. With a little experience, one can select coefficients

for different water-sheds with considerable accuracy. On the fifteen streams cited later, for example, the writer's first choice of coefficients never varied from the final choice by more than one-tenth and seldom by more than one-half a tenth. The final coefficient was adopted after the computed precipitation minus losses for 3 or 4 years had been checked against the observed run-off.

The Transpiration Curve.

The base values for total transpiration, in inches of depth, during the growing season on any given water-shed, are selected with reference to the character of the vegetation and the length of the growing season on that water-shed, giving consideration also to available sunshine. In the following computations a normal seasonal transpiration of about 9 in. has been assumed for small grains, grasses, and other agricultural crops, 8 in. for deciduous trees, 4 in. for evergreen trees, and 6 in. for small trees and brush. The normal monthly distribution of this total seasonal transpiration is based mainly on temperature. To obtain actual transpiration in any given month, however, the values taken from the transpiration curve, after being multiplied by a coefficient, must be further modified on the basis of available moisture. Where precipitation minus evaporation for a given month is insufficient to meet the normal plant requirements for that month, the ground-water is drawn on to a varying extent, depending on the character of the root system of the given vegetation, the depth and character of the soil, and the quantity of surface soil storage, as determined by the precipitation minus losses for previous months.

Computing Annual Run-Off.

The main features of the writer's method of computing run-off to supplement observed stream-flow data may be summarized as follows:

I.—Collection of physical data.

A.—Rainfall and temperature data for stations on and near the given water-sheds from which monthly rainfall and temperature for the water-shed are estimated. Rates of excessive rainfall at different seasons of the year, as indicated by Weather Bureau observations at the nearest regular station. In case rainfall and temperature data are meager,

charts showing isotherms and isohyetals for the portion of the State in which the water-shed is situated are of assistance.

B.—Data relating to wind velocity, relative humidity, and any other prominent weather characteristics.

C.—Data relating to topography, vegetal cover, soil and subsoil, as affecting evaporation.

D.—Data relating to character and density of vegetation and length of growing season, with reference to temperature and hours of sunshine.

E.—Data relating to area of open water surfaces, swamps, and marshes.

II.—Determination of losses.

A.—Evaporation from water area.

1.—Monthly evaporation corresponding to given temperature and season, taken off curve, Fig. 12, and multiplied by percentage of water surface, based on data under *I-E,* and coefficient based on data under *I-B.*

B.—Evaporation from land area.

1.—Determination of coefficient for given water-shed, based principally on physical data under *I-C* and *I-B.*

2.—Determination of evaporation, in inches depth per month, corresponding to given monthly temperature and rainfall for given season of year, from curve of evaporation from land areas, Fig. 19, and multiplication of the same by percentage of land area and coefficient determined under *II-B-1.*

C.—Transpiration from land area.

1.—Determination of normal seasonal transpiration, based on physical data under *I-D.*

2.—Determination of transpiration coefficient by finding ratio between seasonal transpiration determined from base curve of transpiration (Fig. 17) for the normal monthly temperatures for the given water-shed, and the normal seasonal transpiration determined under *II-C-1.*

3.—Determination of monthly transpiration by applying transpiration coefficient to monthly values taken off transpiration curve for given monthly temperatures, and modification of these monthly values on basis of rainfall, percolation, and storage.

III.—Determination of total loss by summation of monthly losses from land and water areas, the deduction of these monthly losses from the monthly precipitation, and summation of these monthly residuals to give the annual yield of the given water-shed, with or without correction of this annual total for fall surface run-off, or changes in ground and surface storage.

IV.—Where the annual yield and its distribution throughout the year are both desired, additional curves similar to those for the Root River water-shed, and computations similar to those given in Table 35, for the same water-shed, must be made. When the more detailed computations, as here indicated, are carried out, it is possible to make more accurate estimates of transpiration during months of deficient rainfall, because more accurate values of soil and subsoil storage are available.

CHARACTERISTIC WATER-SHEDS.

In order to show the practical application of the writer's method of computing run-off, water-sheds of widely different characteristics have been selected, varying in location from Virginia to California, and from Minnesota to Texas.

Table 5 gives the coefficients of evaporation from land areas for each of the fifteen water-sheds considered, together with the principal observed and computed physical data pertaining to each.

DESCRIPTION OF WATER-SHEDS.

In order to show the relation between evaporation coefficients, transpiration, and water-shed characteristics, a brief description of the fifteen water-sheds, giving the facts on which the choice of coefficients is mainly based, follows. It is realized that the information given is meager, and to those intimately familiar with the various water-sheds, perhaps not entirely accurate. Whatever discrepancies exist, however, would indicate that reasonably accurate computed values

of run-off can be secured by the writer's method, notwithstanding this fact. When reasonably good rainfall and temperature data are available, considerable latitude in the choice of coefficients is allowable.

TABLE 5.—OBSERVED AND COMPUTED PHYSICAL DATA FOR FIFTEEN WATER-SHEDS.

Name of water-shed.	Years of record.	Evaporation coefficient.	Area, in square miles.	Observed and Computed Physical Data—Mean Annual.							
				Rainfall.	Temperature.	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	Computed run off.	Observed run-off.
Mississippi	17	1.20	19 500	27.3	41	14.4	7.7	22.1	5.2	5.23	5.31
Little Fork	5	1.10	1 720	23.9	37	11.2	6.9	18.1	5.8	5.80	....
										*5.15	*5.18
Minnesota	5	1.25	6 300	22.7	43	14.1	7.5	21.6	1.1	1.1	....
									*0.77	*0.77	*0.70
Root	6	1.225	1 560	31.2	45	16.5	8.6	25.1	6.1	6.10	....
										*5.40	*5.20
Ottertail	6	1.10	1 310	23.0	40	13.5	6.6	20.1	2.9	2.80	....
										†2.66	†2.59
St. Croix	11	1.05	5 930	30.0	41	13.1	7.0	20.1	9.9	9.90	9.60
Ohio	14	0.875	23 820	41.1	51	14.8	5.8	20.6	20.5	20.50	22.00
Tohickon Creek	24	0.90	102	48.9	51	16.7	7.0	23.7	25.2	25.2	26.10
James	7	0.925	6 230	42.1	54	16.3	7.0	23.3	18.8	18.90	18.00
Roanoke	9	0.90	390	42.6	57	16.9	7.0	23.9	18.7	18.60	17.70
Tombigbee	9	1.05	4 440	49.2	62	22.8	8.4	31.2	18.0	18.00	17.10
Colorado	10	1.20	37 000	26.9	66	17.7	8.1	25.8	1.1	1.06	0.74
Sacramento	9	0.85	10 400	32.2	52	8.5	2.4	10.9	21.3	21.3	20.40
Pit	6	1.10	2 950	14.8	48	6.9	3.0	9.9	4.9	4.9	....
									*3.87	*3.87	*3.92
McCloud	6	0.60	608	61.9	55	8.2	2.4	10.6	51.3	51.3	54.00

* Four years' records.

† Five years' records.

In the State of Minnesota.

Mississippi River Water-Shed, at Minneapolis.—Practically the entire water-shed is heavily covered with glacial drift, and considerable portions are distinctly sandy. Elevations along the river vary from about 800 to 1 475 ft., making the average fall a little less than 1½ ft. per mile.

Although considerable lake and swamp area is known to exist, particularly in the northern part of the State, no allowance has been made for loss from water surface in the computations, except that the coefficient of evaporation has been increased slightly.

Only a comparatively small portion of the water-shed in the northern part of the State is still covered with dense forests. Much of the area has been cut over and is now grown up to birch, poplar,

and other brush; there is a scattering growth of jack and other pines. Perhaps one-third of the area may be considered under cultivation.

Little Fork River Water-Shed, at Little Fork, Koochiching County. —This water-shed lies in the extreme northern part of the State, and is characterized by distinctly flat topography, with only a few hills rising more than 50 or 75 ft. above the plain. The general slope is insufficient to provide good drainage, consequently there are extensive areas of swamp.

The soil is quite clayey. Very little land is under cultivation. Although logging has been carried on extensively for some years, the cut-over area has quickly grown up with poplar, birch, jack and other pines, so that the water-shed, in general, must be considered as densely wooded.

Elevations in the basin range from about 1 100 to 1 400 ft., except for a narrow strip at the head-waters in the Iron Ranges, where elevations run up to 1 700 ft. and higher. The slope along the main stream averages about 2 ft. per mile.

Minnesota River Water-Shed, at Montevideo, Chippewa County.— This water-shed lies in the western part of the State, and a very large portion of it is flat open prairie. Almost all of it is under cultivation. Near the head-waters there are large areas of marsh on clayey soil that contribute little run-off. Although the main stream lies in a valley which is from 100 to 200 ft. below the general level of the water-shed, most of the tributary streams run in comparatively shallow valleys, except for V-shaped gullies which they have cut through the bordering bluffs as they approach the main stream. There are ridges with steep slopes over a portion of the southern and southwestern side of the water-shed. The slope of the river in the reaches above Big Stone Lake is about 20 ft. per mile. Here the bed is frequently dry. From Big Stone Lake to the Montevideo Station the slope averages only about 0.6 ft. per mile.

Transpiration is limited mainly by the quantity of available water, there being a deficiency during almost every growing season. Transpiration is assumed over the entire water-shed, even though 5% is estimated to be lake and open marsh, because of the fact that the excessive transpiration of the long marsh grasses more than compensates for the lack of transpiration over lake areas.

The run-off from this water-shed is so extremely small that it was believed desirable to take into consideration losses from water areas. The exact area of lake and open marsh was not known, but was estimated to be about 5 per cent.

Root River Water-Shed, at Houston, Houston County.—This water-shed lies in the southeastern corner of the State. The topography varies from relatively flat to gently undulating. The streams, however, almost all run in deep V-shaped valleys, that of the main stream being several hundred feet below the general level of the surrounding country.

Most of the land is under cultivation. Timbered areas are practically confined to the watercourses.

The soil is largely clayey loam, rather impervious, subjecting the precipitation to high evaporation losses. The slope of the water-table to the deep drainage courses is steep, even though the water-table lies far below the surface of the ground.

The average slope along the Root River is about 6 ft. per mile.

Ottertail River Water-Shed, at Fergus Falls, Ottertail County.—The entire water-shed lying to the west of the center of the State, and just north of that of the Minnesota River, previously described, is dotted with lakes of various sizes, and many have no outlets. Others have outlets only in high water. It is estimated that the water surfaces aggregate 15% of the entire area.

The topography is prominently rolling, morainic, and knolly, a portion at the southern extremity being more gently undulating. The entire area is heavily covered with drift, varying from clays to sand, gravel, and stony moraines.

The water-shed, on the whole, is lightly timbered, with considerable land under cultivation in the southern portion. Good natural storage is manifested by well-sustained stream flow during the winter and during dry seasons.

In the States of Minnesota and Wisconsin.

St. Croix River Water-Shed, at St. Croix Falls, Polk County, Wis.—This water-shed lies on both sides of the eastern boundary and nearly opposite the center of the State. The topography, in general, is undulating. Practically the entire basin is thickly covered with glacial drift, much of it sand and gravel. There are no rock outcrops.

except near the streams. The northern portion is distinctly rolling. Part of the southern portion is quite flat. In the southeast corner there is an area consisting of knolls and basins.

There are virtually no marshes on the water-shed. In the Wisconsin portion, however, the number of lakes is considerable and many of them have no outlet. Good natural ground storage is shown by the stream flow.

Elevations along the river vary from 750 to 1 000 ft., the average slope of the stream being $2\frac{1}{2}$ ft. per mile.

In the State of West Virginia.

Ohio River Water-Shed, at Wheeling.—The Ohio River is formed by the junction of the Allegheny and the Monongahela, at Pittsburgh, Pa., about 90 miles above Wheeling.

The Allegheny River Basin constitutes about two-thirds of the entire area, and the Monongahela the remaining third. The northeastern portion is rolling and hilly. Practically the entire remaining portion of both the Allegheny and the Monongahela water-sheds are mountainous. The depth of soil, on the whole, is small. The slope of the Allegheny is about $6\frac{1}{2}$ ft. per mile.

Winter conditions in the upper part of the Allegheny Basin are severe, snowfall being heavy and ice forming to considerable thickness.

The Monongahela rises in West Virginia, and flows in a generally northerly direction to its junction with the Allegheny at Pittsburgh.

The upper portion of the water-shed is mountainous; the slopes of the valleys are steep, and in many places precipitous. The lower portion of the river valley, though less mountainous, is still very rolling, and the soil is quite rocky, with but little power of absorption.

Most of the water-shed of the Ohio above Wheeling consists of cut-over land, which formerly was heavily forested, largely with hardwood timber.

Ice conditions are not very severe, due to the southern latitude, and the snowfall, except in the mountains, is comparatively light.

In relative humidity, the Ohio River water-shed, above Wheeling, and the Mississippi, above Minneapolis, are approximately the same, but the average wind velocity in the Ohio Basin is very much less than that in the Mississippi.

In the State of Pennsylvania.

Tohickon Creek Water-Shed, at Point Pleasant.—This water-shed is in Bucks County, north of Philadelphia. The Creek flows eastward into the Delaware River.

According to Hering, in 1885, 76 sq. miles were cultivated and improved land, and 26 sq. miles were untillable and wooded.

The fall of the stream is about 20 ft. per mile. The slopes in the basin are very steep.

In the State of Virginia.

James River Water-Shed, at Cartersville.—The James River rises in the Allegheny Mountains, and the Station of Cartersville lies on a plateau about 50 miles above Richmond.

The portion of the water-shed in the Allegheny Mountains is very broken and has steep slopes. On the Piedmont Plateau the topography is rolling, and the uplands are rounded, with consequently a small range in elevation. Elevations in the head-waters run up to 4 000 ft.

The mountain sides are forested, but a portion of the lower water-shed is under cultivation.

Roanoke River Water-Shed, at Roanoke.—The Roanoke River rises among the eastern foot-hills of the Appalachian Mountains, and flows southeastward to Albemarle Sound, in North Carolina.

The topography above Roanoke is rugged, the soil cover scant, and little of the land is under cultivation.

The range of altitude above Roanoke is about 2 000 ft. The slopes along the stream are steep.

In the State of Mississippi.

Tombigbee River Water-Shed, at Columbus.—The country, on the whole, is flat. The streams have a gentle uniform slope. Hardly one-quarter of the water-shed has been improved. The remainder is still heavily forested. The soil is deep and, for the most part, is a rather heavy loam. The humidity is not much higher than that of the Northwest. An average of about 4 in. of snow falls annually on this water-shed.

In the State of Texas.

Colorado River Water-Shed, at Austin.—This water-shed is comparatively flat, but the slopes are rather pronounced. Timber is found

along the watercourses and at the higher elevations only. The soil is deep, and not especially sandy.

The humidity and wind velocity both differ but slightly from those prevailing throughout most of the Northwest.

In the State of California.

Sacramento River Water-Shed, at Red Bluff, Including its Tributaries, the Pit and the McCloud.—The Sacramento River drains the basin between the Coast and Trinity Ranges on the west, and the Sierra Nevadas on the east. The western slope constitutes a very narrow belt. The main portion of the discharge is derived from the Pit River Basin, which has its head-waters in the Warner Mountains in the extreme northeastern part of the State.

Perhaps half of this water-shed is covered with timber. The other half supports a fair growth of grass, and is used extensively for pasturage. The soil covering the underlying lava is quite scant. Most of the precipitation, particularly over the head-waters of the stream, is in the form of snow.

The Pit River Basin comprises that portion of the Sacramento River water-shed lying to the northeast. It is relatively flat and with little timber growth, most of it being pasture land. Most of the area above Bieber lies at an elevation of from 4 000 to 5 000 ft. above sea level.

The McCloud River enters the Pit from the north a short distance above the place where the latter discharges into the Sacramento. It drains the southern slope of Mt. Shasta. Most of the water-shed is at an elevation ranging between 3 000 and 5 000 ft. The slopes are very steep and practically all the minor tributaries rise among mountain peaks. A very large proportion of the precipitation is snow, which gradually melts and feeds the stream during the summer when the precipitation is practically nil.

It was impossible to obtain either temperature or precipitation records directly on the McCloud River water-shed. For portions of the time, data were available for the Stations of Sisson, Dunsmuir, Delta, Shasta, and Redding, on the Sacramento just west of the McCloud. The precipitation data for these stations are believed to give approximately correct values for the McCloud River water-shed. The temperatures, based on the observed data at these stations, however, are believed to give too high values.

APPLICATION OF METHOD TO CHARACTERISTIC WATER-SHEDS.

It is believed that the diagrams, Figs. 20 to 29, for ten of the fifteen water-sheds to which the method has been applied, indicate clearly the inter-relationship between temperature, monthly distribution of rainfall, and run-off, irrespective of differences in the physical characteristics of the several water-sheds.

Three distinct types of monthly rainfall distribution are shown. The first is characterized by a practically uniform distribution of rainfall throughout the year, as exemplified by Tohickon Creek in particular. A somewhat similar distribution of rainfall prevails on the water-sheds of the Ohio, the James, the Roanoke, and the Tombigbee Rivers.

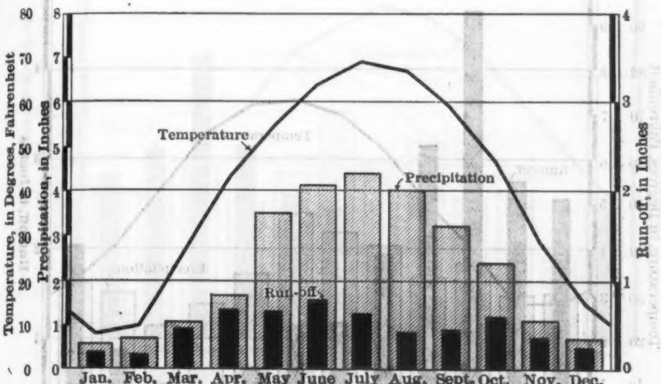
With the exception of the Colorado and the Tombigbee, the mean summer temperatures do not differ greatly on the several water-sheds shown. The mean winter temperatures, however, differ quite widely, varying from about 10° on Minnesota water-sheds to about 30° on those in Pennsylvania, Virginia, West Virginia, and California, and nearly 50° for the Colorado River water-shed in Texas and the Tombigbee in Mississippi.

In the second type of rainfall distribution, the precipitation is high during the summer, and low during the winter, as exemplified by the St. Croix, the Mississippi, and the Colorado River water-sheds.

This type of rainfall distribution results in proportionately large evaporation losses, in both summer and winter. Although the winter evaporation losses average only 0.2 or 0.3 in. per month in Minnesota, they are proportionately large because the precipitation averages less than 1 in. per month during this period. The high summer precipitation results in heavy evaporation and transpiration losses, on account of high temperatures.

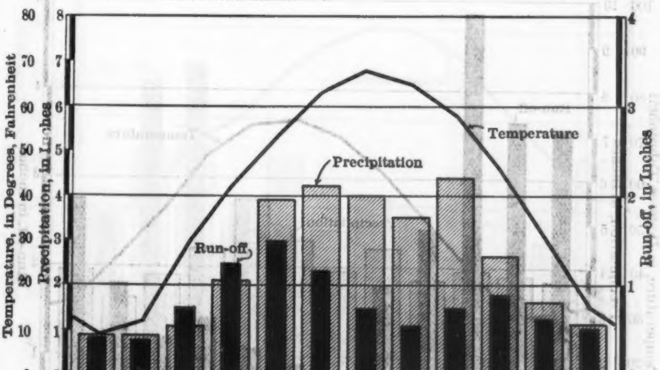
It is interesting to note that if the summer precipitation of about 4 in. per month on the St. Croix and Mississippi River water-sheds occurred at temperatures prevailing on the Colorado River water-shed, the losses out of precipitation due to temperature alone would reduce the run-off from these water-sheds to approximately the same as that observed on the Colorado.

Similarly, if the winter precipitation of about 1 in. per month occurred at the Texas winter temperatures of nearly 50°, instead of at



MISSISSIPPI RIVER WATER-SHED
MINNEAPOLIS, MINN.
AREA=19 500 SQ.MILES
1897-1913.

FIG. 20.



ST.CROIX RIVER WATER-SHED
ST.CROIX FALLS, WIS.
AREA=5930 SQ.MILES
1901-1912

FIG. 21.

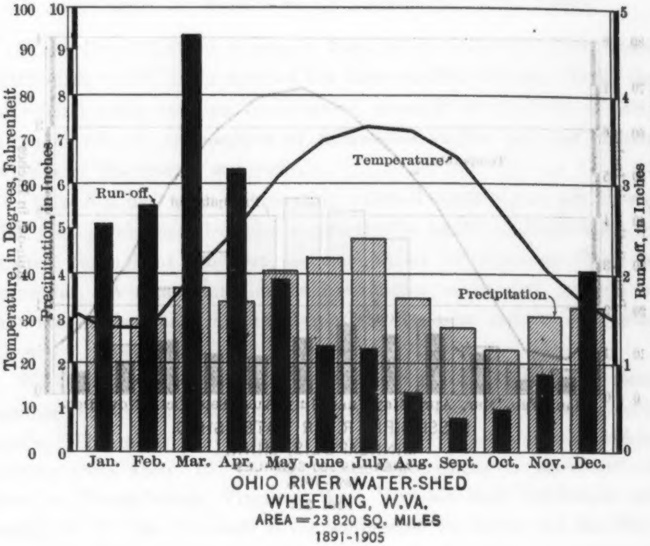

Temperature, in Degrees, Fahrenheit
Precipitation, in Inches
Run-off, in Inches
Run-off
Temperature
Precipitation
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.
OHIO RIVER WATER-SHED
WHEELING, W.VA.
AREA = 23 820 SQ. MILES
1891-1905

FIG. 22.

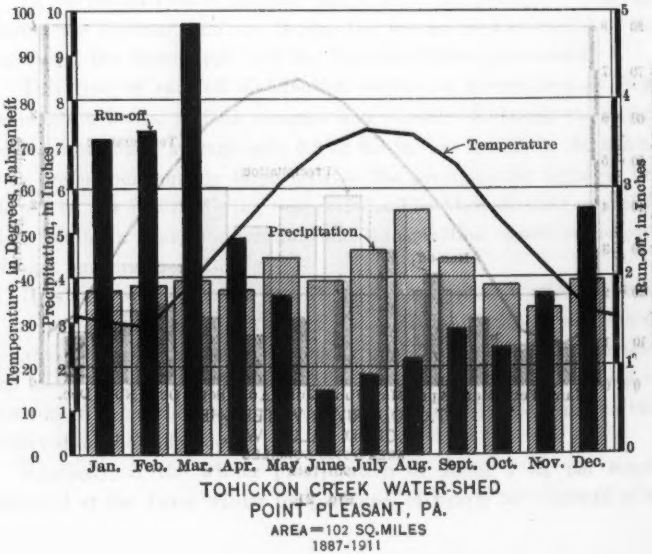

Temperature, in Degrees, Fahrenheit
Precipitation, in Inches
Run-off, in Inches
Run-off
Temperature
Precipitation
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.
TOHICKON CREEK WATER-SHED
POINT PLEASANT, PA.
AREA = 102 SQ. MILES
1887-1911

FIG. 23.

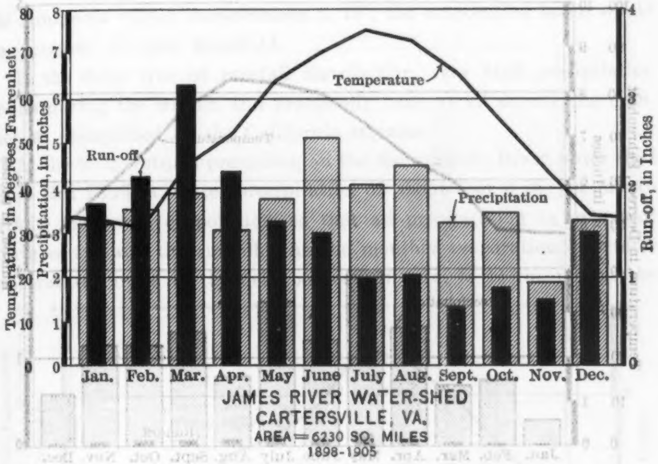

Temperature
Run-off
Precipitation
Temperature, in Degrees, Fahrenheit
Precipitation, in Inches
Run-off, in Inches
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.
JAMES RIVER WATER-SHED
CARTERSVILLE, VA.
AREA = 6230 SQ. MILES
1898-1905

FIG. 24.

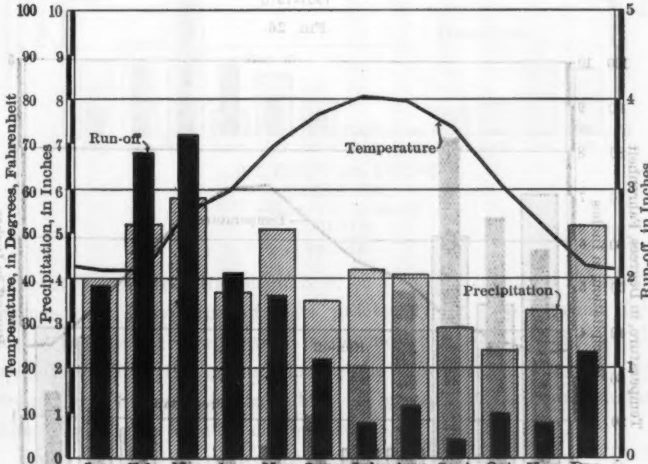
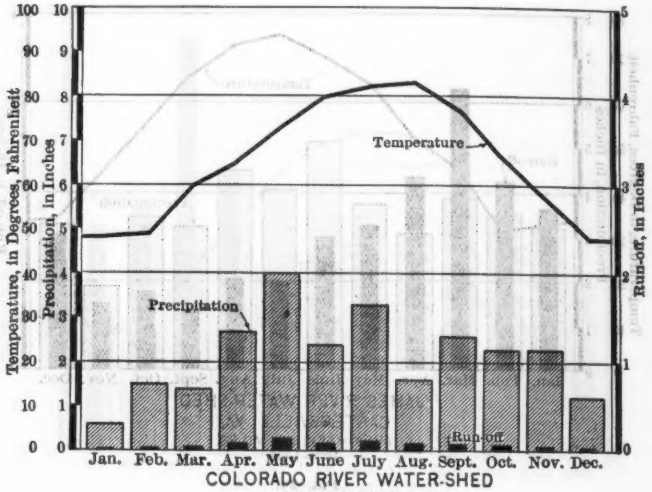
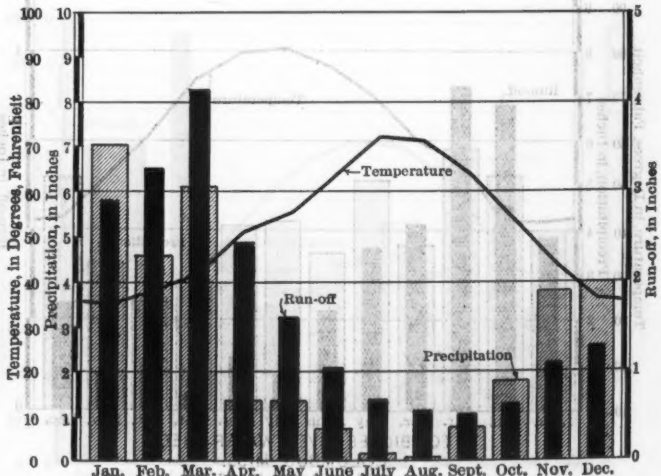

Run-off
Temperature
Precipitation
Temperature, in Degrees, Fahrenheit
Precipitation, in Inches
Run-off, in Inches
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.
TOMBIGBEE RIVER WATER-SHED
COLUMBUS, MISS.
AREA = 4440 SQ. MILES
1900-1909

FIG. 25.



COLORADO RIVER WATER-SHED
AUSTIN, TEXAS
AREA = 37 000 SQ. MILES
1901-1910

FIG. 26.



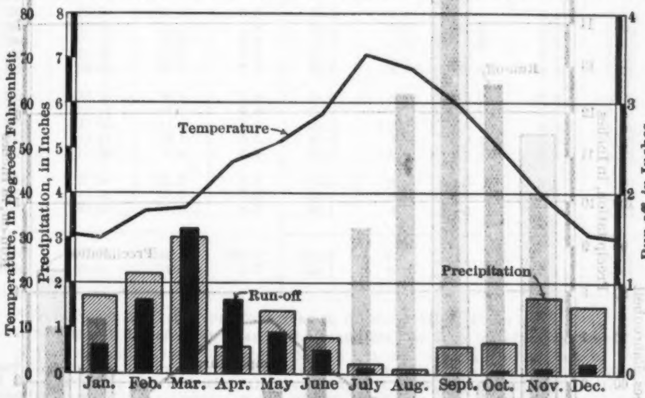
SACRAMENTO RIVER WATER-SHED
RED BLUFF, CAL.
AREA = 10 400 SQ. MILES
1902-1911

FIG. 27.

the Minnesota winter temperatures of 10°, the evaporation losses would be increased at least threefold.

In the third type of rainfall distribution, very high precipitation occurs during the winter, and practically none at all during the summer, as exemplified by the California streams.

At the temperatures prevailing on the Sacramento River water-shed above Red Bluff, a winter precipitation of about 5 or 6 in. per month permits maximum evaporation, so that all precipitation in excess of this goes to swell the run-off. As the monthly evaporation at winter temperature ranges in the neighborhood of 1 in. to 1½ in., it must be



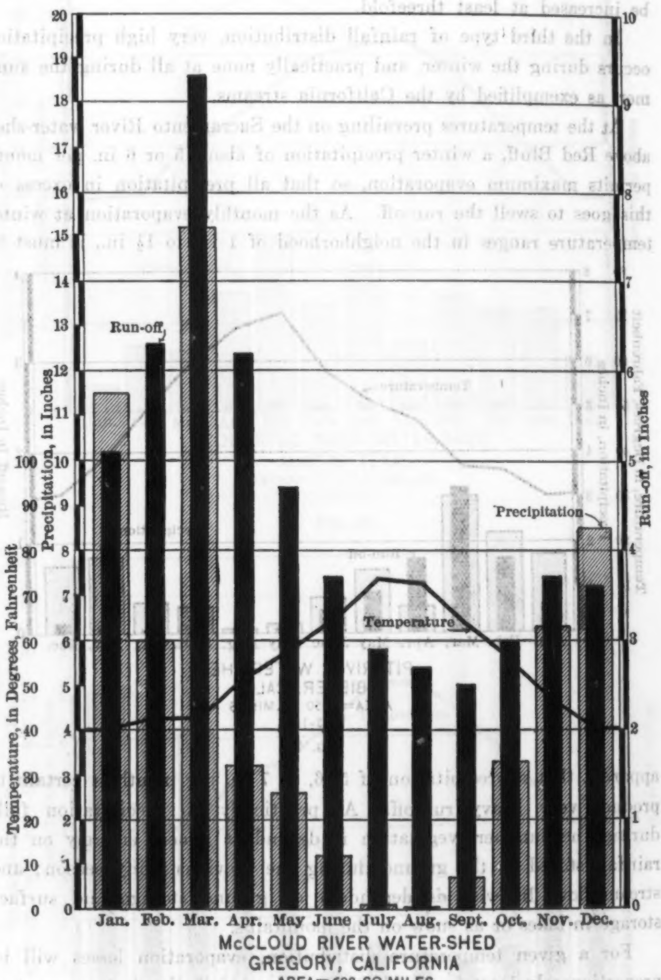
PIT RIVER WATER-SHED
BIEBER, CAL.
AREA=2950 SQ.MILES
1903-1908

FIG. 28.

apparent that a precipitation of 5, 6, or 7 in. per month is certain to produce very heavy run-off. As practically no precipitation falls during the summer, vegetation is dependent almost entirely on the rainfall stored in the ground during the previous rainy season; and stream flow, likewise, is dependent on ground storage and surface storage in lakes or as snow on the mountains.

For a given temperature distribution, evaporation losses will be proportionately least for the last type of rainfall distribution, greatest for the second type, and intermediate for the first type.

The necessity of taking into account monthly temperatures in any attempt at computing run-off, may bear emphasis. The mean tem-



McCLOUD RIVER WATER-SHED
GREGORY, CALIFORNIA
AREA=608 SQ.MILES
1902-1908

FIG. 29.

TABLE 6.—SUMMARY OF DATA AND COMPUTATIONS FOR THE MISSISSIPPI RIVER WATER-SHED, ABOVE MINNEAPOLIS, MINN.

Area = 19 500 sq. miles.

Year.	Rainfall.*	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	Surface run-off.	Computed run-off.	Observed run-off.†
1897	32.58	15.6	8.8	24.4	8.2	0	8.2	8.16
1898	24.29	13.4	7.5	20.9	3.4	0	3.4	3.85
1899	32.54	16.3	8.8	25.1	7.4	0	7.4	6.64
1900	28.14	16.2	8.4	24.6	3.5	0	3.5	3.82
1901	22.57	12.4	6.8	19.2	3.4	0	3.4	3.31
1902	25.80	14.7	7.8	22.5	3.3	0.2	3.5	3.30
1903	32.85	16.3	8.5	24.8	8.0	—0.2	7.8	6.41
1904	26.59	13.4	7.8	21.2	5.4	0	5.4	5.45
1905	32.68	16.7	7.8	24.5	8.2	0.6	8.8	9.71
1906	32.48	16.1	7.9	24.0	8.5	—0.2	8.3	9.28
1907	24.46	12.0	6.0	18.0	6.5	—0.2	6.3	6.68
1908	27.60	14.8	7.2	22.0	5.6	0	5.6	6.22
1909	24.04	13.1	6.7	19.8	4.2	0.1	4.3	5.23
1910	17.96	9.4	6.0	15.4	2.6	—0.3	2.3	2.33
1911	27.54	15.5	8.5	24.0	3.5	0	3 5	2.20
1912	23.01	13.0	7.0	20.0	3.0	0	3.0	3.52
1913	28.83	15.9	8.5	24.4	4.4	0	4.4	4.23
Total..	463.96	244.8	130.0	374.8	89.1	....	89.0	90.34
Mean..	27.3	14.4	7.7	22.1	5.2	....	5.2	5.31

* From November 1st of previous year to October 31st of given year.

† From March 1st of given year to February 28th or 29th of following year.

NOTE.—A run-off year, beginning March 1st and ending on the last of February following, is assumed to correspond with the rainfall year beginning November 1st and ending on the following October 31st. It is assumed that practically all excess ground-water available for run-off on November 1st of any year will have appeared as run-off before the following March 1st. Where the temperature for November was high, an estimate has been made of the precipitation which appeared immediately as surface run-off, instead of being held through the winter as snow and added to the run-off of the following year.

It may be noted that during 1911 the discharge of most of the Minnesota streams was exceptionally low, even though the rainfall was above the average. This can be accounted for by the fact that a large portion of the precipitation was used to replenish excessive depletion of ground storage during 1910, which was the dryest year on record for Minnesota.

perature for April, on the Mississippi River water-shed, for example, varied in 17 years from 32.2° in 1907 to 48.8° in 1910. Assuming a precipitation of 3 in. per month, this change in temperature would result in a variation in the evaporation loss of about 0.4 in., besides a loss due to transpiration of 0.2 or 0.3 in., making a total possible difference in run-off of from 0.6 to 0.7 in., resulting from the observed temperature variation for this one month alone. An equal difference in evaporation loss is indicated by the variation in May temperatures from 44.9° in 1907 to 60.3° in 1911. This variation in temperature

TABLE 7.—DATA AND COMPUTATIONS FOR THE MISSISSIPPI RIVER WATER-SHED.

Year and month.	Monthly temperature, in degrees Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area. Transpiration, in inches.	Evaporation, in inches. From curve.	Evaporation, in inches. Actual.	Total loss, in inches.	Precipitation minus total loss, in inches.
1901.							
Nov.....	28.5	0.97	....	0.20	0.24	0.24	0.73
Dec.....	13.8	0.55	....	0.30	0.36	0.36	0.19
1902.							
Jan.....	14.8	0.37	....	0.35	0.42	0.42	−0.05
Feb.....	15.6	0.50	....	0.35	0.42	0.42	0.08
Mar.....	33.6	0.72	....	0.50	0.60	0.60	0.12
April...	41.7	1.45	0.1	0.75	0.90	1.00	0.45
May....	57.1	5.32	1.3	2.25	2.70	4.00	1.32
June....	62.1	3.28	1.6	1.70	2.04	3.64	−0.36
July....	69.9	3.96	2.0	2.20	2.64	4.64	−0.68
Aug.....	65.2	4.76	1.7	2.12	2.54	4.24	0.52
Sept	55.1	1.69	0.8	0.75	0.90	1.70	−0.01
Oct.....	46.7	2.23	0.3	0.75	0.90	1.20	1.03
	....	25.80	7.8	12.22	14.66	22.46	3.34
1906.							
Nov.....	31.0	2.15	....	0.45	0.54	0.54	1.61
Dec	14.9	0.93	....	0.30	0.36	0.36	0.57
1907.							
Jan.....	1.5	1.22	....	0.08	0.10	0.10	1.12
Feb.....	14.3	0.53	....	0.30	0.36	0.36	0.17
Mar.....	26.5	0.92	....	0.50	0.60	0.60	0.32
April...	32.2	0.80	....	0.50	0.60	0.60	0.20
May....	44.9	2.82	0.3	*1.00	1.20	1.50	1.32
June....	63.9	3.23	1.6	*1.60	1.92	3.52	−0.29
July....	68.7	2.63	1.6	*1.45	1.74	3.34	−0.71
Aug.....	66.5	4.14	1.5	*1.90	2.28	3.78	0.36
Sept.....	56.2	3.83	0.9	1.50	1.80	2.70	1.13
Oct.....	45.4	1.26	0.1	0.45	0.54	0.64	0.62
	....	24.46	6.0	10.03	12.04	18.04	6.42

* Low storage.

NOTE.—The column for actual evaporation represents values in the preceding column multiplied by an evaporation coefficient for the given water-shed and a percentage of the land area.

would also result in a variation in transpiration loss of at least 1 in., or a total difference in annual run-off of about 25%, resulting from the observed variation in temperature for this one month alone.

Similar variations occur in monthly summer temperatures. August, 1903, for example, had a temperature of 62.7° as compared with 74.1°

TABLE 8.—SUMMARY OF DATA AND COMPUTATIONS FOR THE LITTLE FORK RIVER WATER-SHED, ABOVE LITTLE FORK, MINN.

Area = 1 720 sq. miles.

Year.	Rainfall.*	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	Surface run-off.	Computed run off.	Observed run-off.†
1909.......	28.4	12.8	7.2	20.0	8.4	0.0	8.4	....
1910........	19.0	8.9	6.3	15.2	3.8	0.0	3.8	3.75‡
1911........	23.4	11.5	7.2	18.7	4.7	0.0	4.7	4.62
1912........	22.0	10.8	6.9	17.7	4.3	0.0	4.3	4.21
1913........	26.9	12.3	6.8	19.1	7.8	0.0	7.8	7.95
Total......	119.7	56.3	34.4	90.7	29.0	....	29.0	20.53
Mean......	23.9	11.2	6.9	18.1	5.8	....	5.8§(5.15)	5.13

* From November 1st of previous year to October 31st of given year.

† From April 1st of given year to March 31st of following year.

‡ Part estimated.

§ Mean of years over which observations extend.

NOTE.—No allowance for changes in storage has been made on this water-shed because the period from the close of the rainfall year to the beginning of the run-off year is 1 month (20% longer than that for the other Minnesota water-sheds considered).

The topography is quite flat, with practically no lake areas, and the soil clayey, so that little capacity for storage is to be expected. This is borne out by the available stream-flow records, which show comparatively little variation in the late winter discharge.

These records also show practically no surface run-off during November.

for August, 1900. Assuming a precipitation of 5 in. per month, the evaporation loss alone, as given by the curve, would vary by 0.8 in. Applying a coefficient of 1.2 would make the variation in the evaporation loss practically 1 in.

Similar changes in temperature occur on other water-sheds. On the Tohickon Creek, for example, within a period of 24 years, the January temperature varied from 19° in 1893 to 38° in 1890. The February temperature varied from 22° in 1895 to 40° in 1909, and the August temperature from 68° in 1889 to 76° in 1900. All these differences in temperature, if not taken into account in computing losses out of rainfall, would make a substantial difference in the computed run-off.

Tables 6 to 34 contain data and computations for fifteen water-sheds, together with detailed computations for characteristic years on each.

TABLE 9.—DATA AND COMPUTATIONS FOR THE LITTLE FORK RIVER WATER-SHED.

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area.			Total loss, in inches.	Precipitation minus total loss, in inches.
			Transpiration, in inches.	Evaporation, in inches.			
				From curve.	Actual.		
1908.							
Nov.....	29	0.8	....	0.20	0.22	0.22	0.58
Dec.....	11	0.8	....	0.20	0.22	0.22	0.58
1909.							
Jan.....	2	1.6	....	0.07	0.08	0.08	1.52
Feb.....	6	0.6	....	0.12	0.13	0.13	0.47
Mar.....	22	0.8	....	0.50	0.55	0.55	0.25
Apr	30	1.0	....	0.60	0.66	0.66	0.34
May....	49	2.6	0.9	1.15	1.27	2.17	0.43
June ...	63	1.9	1.6	1.15	1.27	2.87	—0.97
July....	67	4.8	1.7	2.25	2.48	4.18	0.12
Aug....	66	7.0	1.7	3.00	3.30	5.00	2.00
Sept....	56	3.4	1.1	1.40	1.54	2.64	0.76
Oct.....	42	3.6	0.2	1.00	1.10	1.30	2.30
	....	28.4	7.2	11.64	12.82	20.02	8.38
Nov.....	29	1.6	....	0.32	0.35	0.35	1.25
Dec.....	8	1.4	....	0.15	0.16	0.16	1.24
1910.							
Jan.....	8	0.4	....	0.17	0.19	0.19	0.21
Feb.....	3	0.5	....	0.08	0.09	0.09	0.41
Mar.....	37	0.4	....	0.45	0.50	0.50	—0.10
Apr	45	2.4	0.2	1.00	1.10	1.30	1.10
May....	49	1.3	0.8	0.70	0.77	1.57	—0.27
June....	67	0.9	1.3	0.70	0.77	2.07	—1.17
July....	67	3.5	1.6	1.90	2.09	3.69	—0.19
Aug....	60	1.3	1.1	0.65	0.72	1.82	—0.52
Sept....	55	3.0	1.1	1.20	1.32	2.42	0.58
Oct.....	46	2.3	0.2	0.80	0.88	1.08	1.22
	....	19.0	6.3	8.12	8.94	15.24	3.76

MONTHLY DISTRIBUTION OF RUN-OFF.

There is such an intimate relationship between the run-off and the physical characteristics of a water-shed, the daily temperature, and the character and rate of precipitation, that only the most detailed analysis can possibly result in even a reasonably accurate estimate of the daily and monthly distribution of the annual run-off.

In comparing daily or monthly computed stream flow with observed data, it is well to keep in mind, however, that both observed and com-

TABLE 10.—SUMMARY OF DATA AND COMPUTATIONS FOR THE MINNESOTA RIVER WATER-SHED, ABOVE MONTEVIDEO, MINN.

Area = 6 300 sq. miles.

Year.	Rainfall.*	Evaporation.		Transpiration.	Total loss.	Precipitation minus total loss.	Observed run-off.†
		Water.	Land.				
1909	23.72	1.49	12.75	7.0	21.24	2.48	‡
1910	17.76	1.78	8.84	6.5	17.12	0.64	0.90
1911	24.83	1.63	14.00	8.3	23.93	0.90	0.27
1912	25.10	1.50	14.45	8.4	24.35	0.75	0.74
1913	22.24	1.62	12.65	7.2	21.47	0.77	0.89
Total...	113.65	8.02	62.69	37.4	108.11	5.54	2.80
Mean...	22.7	1.6	12.5	7.5	21.6	1.1 § (0.77)	0.70

* From November 1st of previous year to October 31st of given year.

† From March 1st of given year to February 28th or 29th of following year.

‡ The run-off at Mankato for this year was 3.4 in. On this basis, the run-off above Montevideo would probably be from 2.25 to 2.75 in.

§ Mean of years over which observations extend.

NOTE.—A run-off year beginning March 1st and ending on the last of February following, is assumed to correspond to the rainfall year beginning November 1st and ending on the following October 31st. It is assumed that practically all excess ground-water available for run-off on November 1st of any year will have appeared as run-off before the following March 1st. As no surface run-off appeared for November during the given years, the computed precipitation minus total loss for this water-shed is directly comparable with the actual run-off for the run-off year.

puted run-off data for any given water-shed are of service only as a basis for estimating the run-off which will probably occur in the future. An identical recurrence of any given combination of meteorological phenomena on any one water-shed is extremely improbable. In view of this fact, the complete daily, and to a large extent the monthly, distribution of run-off, is of much less importance than the annual yield of a water-shed, the probable extreme maximum flow, the extreme minimum, and a reasonably accurate estimate of run-off below the limit of economical utilization for whichever purpose the stream flow is to be used.

Inasmuch as the low-water flow from most small water-sheds is so extremely small as to be hardly capable of economical use except through storage reservoirs, the sudden fluctuations in stream flow below the maximum expected flood, with the reservoir filled, are of little consequence. Whether 1 in. of run-off occurs in a few days or in a few weeks is not of much consequence on such a water-shed, if all the available run-off can be held in the storage reservoir for gradual

TABLE 11.—DATA AND COMPUTATIONS FOR THE MINNESOTA RIVER WATER-SHED.

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Loss from Water Area. Evaporation, in inches.		Loss from Land Area.			Total loss, in inches.	Precipitation minus total loss, in inches.
			From curve.	Actual.	Transpiration equivalent, in inches.	Evaporation, in inches. From curve.	Evaporation, in inches. Actual.		
1908.									
Nov.........	35.5	1.89	....	....	....	0.42	0.52	0.52	1.37
Dec.........	18.0	1.38	....	....	....	0.40	0.50	0.50	0.88
1909.									
Jan	11.5	0.88	....	....	....	0.25	0.31	0.31	0.57
Feb	14.2	1.49	....	....	....	0.30	0.37	0.37	1.12
Mar	28.0	0.25	....	....	....	0.45	0.56	0.56	−0.31
Apr.........	37.5	0.87	2.20	0.11	....	0.55	0.65	0.76	0.11
May.........	55.0	4.81	4.20	0.21	1.20	2.00	2.37	3.78	1.03
June.........	66.2	2.70	5.60	0.28	1.70	1.50	1.78	3.76	−1.06
July.........	70.0	2.21	5.80	0.29	1.20	1.40	1.66	3.15	−0.94
Aug.........	73.2	3.54	5.90	0.30	1.60	1.90	2.25	4.15	−0.61
Sept.........	61.0	1.84	4.00	0.20	1.00	0.90	1.07	2.27	−0.43
Oct.........	45.5	1.86	2.10	0.10	0.30	0.60	0.71	1.11	0.75
	....	23.72	29.80	1.49	7.00	10.67	12.75	21.24	2.48
Nov.........	33.0	1.32	1.10	0.06	....	0.30	0.36	0.42	0.90
Dec.........	10.0	1.75	....	....	....	0.20	0.25	0.25	1.50
1910.									
Jan.........	13.5	0.76	....	....	....	0.30	0.37	0.37	0.39
Feb.........	8.5	0.49	....	....	....	0.25	0.31	0.31	0.18
Mar	45.0	0.25	3.00	0.15	0.1	*0.15	0.18	0.43	−0.18
Apr	49.0	2.28	3.50	0.17	0.2	*0.85	1.02	1.39	0.89
May.........	53.5	0.92	4.00	0.20	0.7	*0.45	0.53	1.43	−0.51
June.........	68.2	2.49	5.80	0.29	1.5	*1.35	1.60	3.39	−0.90
July	72.3	1.49	6.30	0.32	1.0	*0.95	1.13	2.45	−0.96
Aug.........	67.2	3.61	5.00	0.25	1.8	*1.65	1.96	4.01	−0.40
Sept.........	60.0	1.01	3.80	0.19	0.8	*0.40	0.48	1.47	−0.46
Oct.........	54.0	1.39	3.00	0.15	0.4	*0.55	0.65	1.20	0.19
	....	17.76	35.50	1.78	6.5	7.40	8.84	17.12	0.64

* Low storage.

utilization. The engineer is usually much more interested in the total run-off from such a water-shed, up to the point of economical utilization, than in the exact distribution of that run-off through the year.

Although the writer does not pretend to be able to compute accurately the daily or even the monthly discharge of a stream, as previously stated, he does believe it to be possible to compute a reasonable

TABLE 12.—SUMMARY OF DATA AND COMPUTATIONS FOR THE ROOT RIVER WATER-SHED, ABOVE HOUSTON, MINN.

Area = 1 560 sq. miles.

Year.	Rainfall.*	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	Computed run-off.†	Observed run-off.‡
1908	34.8	19.0	8.9	27.9	6.9	....	....
1909	33.2	17.3	9.0	26.3	6.9	4.32§	4.10§
1910	23.2	11.2	7.7	18.9	4.3	4.45	4.22
1911	35.0	18.4	9.2	27.6	7.4	5.28	5.19
1912	29.9	15.6	9.5	25.1	4.8	5.82	5.68
1913	31.9	17.3	9.1	26.4	5.5	5.08	5.73
Total.	188.0	98.8	53.4	152.2	35.8	20.63	20.82
Mean.	31.4	16.5	8.9	25.4	6.0	5.16	5.20

* From November 1st of previous year to October 31st of given year.

† Sum of monthly run-off from March 1st of given year to February 28th or 29th of following year, as computed in Table 35.

‡ From March 1st of given year to February 28th or 29th of following year.

§ From June to February, inclusive.

NOTE.—This water-shed is in the southeastern corner of Minnesota, and the winter temperatures are not sufficiently low to prevent surface run-off, consequently the annual precipitation minus losses figured for the rainfall year beginning Nov. 1st does not check so well with the observed run-off for the year beginning March 1st, as it does on more northerly water-sheds. It will be noted, however, that the computed run-off checks the observed run-off very closely.

distribution of the annual yield of a given water-shed through the various months of the year by making a proper study of the factors that influence this distribution of run-off. To do this requires considerable expenditure of time and the use of data seldom readily available.

In the first place, it is necessary to have detailed knowledge of the physical characteristics of the water-shed under consideration, including topography, geology, vegetation, soil, etc., and the character and daily rate of precipitation. It is important to know whether a given winter's snowfall will run off suddenly during a heavy spring rain, or whether it will melt gradually and be largely absorbed by the soil. It is important to know whether, after heavy rains, a large portion of the precipitation immediately appears in the streams or whether it is paid out gradually from lakes and swamps. It is important to know whether, during the summer, when the demands of evaporation and transpiration absorb most of the available rainfall,

TABLE 13.—SUMMARY OF DATA AND COMPUTATIONS FOR THE OTTERTAIL RIVER WATER-SHED, ABOVE FERGUS FALLS, MINN.

Area = 1 310 sq. miles.

Year.	Rainfall.*	Evaporation.		Transpiration.	Total loss.	Precipitation minus total loss.	Change in storage.	Computed run-off.	Observed run-off.†
		Water.	Land.						
1908.......	25.1	3.4	10.9	6.8	21.1	4.0	0	4.0	3.84‡
1909.......	25.4	3.6	10.9	6.9	21.4	4.0	0.2	3.8	3.90§
1910.......	15.5	3.7	5.9	4.9	14.5	1.0	—0.8	1.8	1.79
1911.......	22.8	3.5	10.1	7.0	20.6	2.2	0.6	1.6	1.48
1912.......	23.6	3.2	11.0	7.2	21.4	2.2	0.1	2.1	1.97
1913.......	25.8	3.4	11.3	7.0	21.7	4.1	0.3	3.8	
Total......	138.2	20.8	60.1	39.8	120.7	17.5	....	17.1‖	12.98
Mean......	23.0	3.5	10.0	6.6	20.1	2.9	....	2.8 (2.66)	2.59

* From November 1st of previous year to October 31st of given year.
† From March 1st of given year to February 28th or 29th of following year.
‡ January and February estimated at 0.34.
§ March estimated at 0.25.
‖ Mean of years over which observations extend.

NOTE.—In view of the large number of lakes on this water-shed, some change in storage is certain to occur, notwithstanding the equalization of ground-water which may be effected during the period between the close of the rainfall year and the beginning of the run-off year.

the stream will be fed by an abundant ground-water storage. It is important to know whether the temperature during the winter suddenly drops to a point where a heavy ice cover is formed over the stream in one or two days, almost shutting off the flow until the stage has sufficiently increased and the slope has been sufficiently equalized and increased, so that the combined area of cross-section and gradient are ample to overcome the increased friction due to ice cover, and thus carry approximately the quantity of water that was flowing in the stream before the freeze-up. It is important to know, further, whether the ground-water table lies so close to the surface that a large portion of the ground-water which would otherwise maintain stream flow during the winter, is frozen up and held until the following spring. It is important to know the absorptive capacity of the soil on a given water-shed, and the character of the underlying soil or rock. It is important to know whether, at a given point, a stream flows over a bed of impervious rock which underlies

TABLE 14.—DATA AND COMPUTATIONS FOR THE OTTERTAIL WATER-SHED.

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Loss from Water Area. Evaporation, in inches. From curve.	Loss from Water Area. Evaporation, in inches. Actual.	Loss from Land Area. Transpiration equivalent, in inches.	Loss from Land Area. Evaporation, in inches. From curve.	Loss from Land Area. Evaporation, in inches. Actual.	Total loss, in inches.	Precipitation minus total loss, in inches.
1907.									
Nov.	29	0.2	1.00	0.15	....	0.04	0.04	0.19	0.01
Dec.	19	0.5	....	....	....	0.22	0.24	0.24	0.26
1908.									
Jan	14	0.2	....	....	....	*0.18	0.20	0.20	0.00
Feb	15	1.6	....	....	....	0.40	0.44	0.44	1.16
Mar.	22	1.5	....	....	....	0.60	0.66	0.66	0.84
Apr	44	1.4	1.70	0.26	0.1	0.70	0.65	1.01	0.39
May.	53	5.2	2.46	0.37	1.0	2.08	1.95	3.32	1.88
June	62	5.5	3.33	0.50	1.5	2.50	2.34	4.34	1.16
July	70	2.6	4.30	0.65	1.5	1.60	1.50	3.65	−1.05
Aug.	65	3.0	3.90	0.58	1.5	1.50	1.40	3.48	−0.48
Sept.	63	2.2	3.70	0.55	1.0	1.15	1.08	2.63	−0.43
Oct	45	1.2	2.10	0.32	0.2	0.41	0.38	0.90	0.30
	....	25.1	22.49	3.38	6.8	11.38	10.88	21.06	4.04
Nov.	32	1.7	1.15	0.17	....	0.35	0.33	0.50	1.20
Dec.	13	0.7	....	....	....	0.27	0.30	0.30	0.40
1909.									
Jan	7	0.9	....	....	....	0.14	0.16	0.16	0.74
Feb	10	0.6	....	....	....	0.21	0.23	0.23	0.37
Mar.	24	0.3	0.88	0.13	....	0.50	0.47	0.60	−0.30
Apr	34	1.5	1.80	0.27	....	*0.60	0.56	0.83	0.67
May.	53	3.9	2.46	0.37	1.0	1.66	1.55	2.92	0.98
June	65	2.7	3.65	0.55	1.5	1.58	1.48	3.53	−0.83
July	67	4.5	4.00	0.60	1.6	2.30	2.15	4.35	0.15
Aug.	71	5.1	4.50	0.68	1.6	2.51	2.35	4.63	0.47
Sept.	59	1.8	3.33	0.50	1.0	0.89	0.83	2.33	−0.53
Oct	43	1.7	1.95	0.29	0.2	0.54	0.50	0.99	0.71
	....	25.4	23.72	3.56	6.9	11.55	10.91	21.37	4.03

* Low storage.

the entire water-shed, thus preventing all possible deep seepage losses, or whether it flows over a bed of sand and gravel overlying pervious sandstone, thus admitting of considerable underflow and deep seepage loss. It is important to know whether the stream flow consists largely of seepage flow, or whether practically all the discharge represents surface run-off resulting from excessive precipitation over restricted areas. These are but a few of the most important considerations which must be kept in mind in estimating the distri-

TABLE 15.—SUMMARY OF DATA AND COMPUTATIONS FOR THE ST. CROIX RIVER WATER-SHED, ABOVE ST. CROIX FALLS, WIS.

Area = 5 930 sq. miles.

Year.	Rainfall.*	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	Surface run-off.	Change in storage.	Computed run-off.	Observed run-off.†
1902	24.66	12.0	6.8	18.8	5.9	0.4	−0.2	6.5	6.65
1903	44.16	16.7	7.7	24.4	19.7	−0.2	2.0‡	17.5	16.14
1904	32.64	13.1	7.1	20.2	12.4	0.2	−0.4	13.0	11.85
1905	32.67	14.5	6.9	21.4	11.3	−0.2	−0.5	11.6	11.73
1906	33.17	14.5	7.0	21.5	11.7	0.0	−0.2	11.9	10.95
1907	29.18	12.1	6.4	18.5	10.7	−0.2	0.6	9.9	10.45
1908	30.57	13.8	6.9	20.7	9.9	0.0	−0.2	10.1	10.02
1909	27.83	12.9	6.7	19.6	8.2	0.4	−0.4	9.0	9.44
1910	18.64	8.0	6.1	14.1	4.5	−0.4	−1.5	5.6	4.86
1911	30.68	14.6	8.1	22.7	8.0	0.0	1.0	7.0	5.61
1912	26.48	12.3	7.2	19.5	7.0	0.0	−0.4	7.4	6.92
Total...	330.68	144.5	76.9	221.4	109.3	....	....	109.5	105.62
Mean..	30.0	13.1	7.0	20.1	9.9	....	....	9.9	9.6

* From November 1st of previous year to October 31st of given year.

† From March 1st of given year to February 28th or 29th of following year.

‡ 24 in. of rainfall from July to October, inclusive.

NOTE.—Large portions of the St. Croix River water-shed are quite sandy, and the topography is such that considerable quantities of water are stored for longer periods than that represented between the close of the rainfall year, November 1st, and the beginning of the next run-off year, March 1st; consequently, for this water-shed, some allowance has been made for a change in storage. Except in one or two years, however, the estimated change in storage has not affected the computed run-off materially. That considerable changes in storage occurred during the years for which such allowance has been made, is substantiated by the variation in actual February run-off.

bution of run-off, as well as in determining the annual yield of a water-shed.

To aid in computing the monthly distribution of annual run-off, diagrams of the type shown in Figs. 30 to 32 for the Root River water-shed have been prepared. These are to be used in conjunction with daily temperature and precipitation records. The curves of Fig. 30 show, in the first place, the approximate maximum quantity of snow which, when available, will melt at the given monthly mean temperatures. The other curves in Fig. 30 give the estimated quantity of this melted snow which will percolate into the ground for various soil storage capacities. By "soil storage capacity" is to be understood, in this case, the approximate quantity of water, in inches of depth, which the upper foot or so of soil is capable of absorbing at

TABLE 16.—DATA AND COMPUTATIONS FOR THE ST. CROIX RIVER WATER-SHED.

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area. Transpiration, in inches.	Evaporation, in inches. From curve.	Evaporation, in inches. Actual.	Total loss, in inches.	Precipitation minus total loss, in inches.
1906.							
Nov.	31.4	2.38	....	0.50	0.52	0.52	1.86
Dec.	17.2	1.34	....	0.40	0.42	0.42	0.92
1907.							
Jan.	5.9	1.47	....	0.10	0.10	0.10	1.37
Feb.	14.0	0.72	....	0.35	0.37	0.37	0.35
Mar.	27.4	1.73	....	0.70	0.74	0.74	0.99
Apr.	34.9	0.60	....	0.50	0.52	0.52	0.08
May	43.7	2.99	0.4	1.15	1.21	1.61	1.38
June	61.8	3.54	1.5	1.75	1.84	3.34	0.20
July	66.4	3.17	1.7	1.78	1.87	3.57	—0.40
Aug.	64.8	2.73	1.5	1.40	1.47	2.97	—0.24
Sept.	55.3	7.77	1.2	2.60	2.73	3.93	3.84
Oct.	43.8	0.74	0.1	0.30	0.32	0.42	0.32
	....	29.18	6.4	11.53	12.11	18.51	10.67
Nov.	21.7	1.75	....	0.43	0.45	0.45	1.30
Dec.	21.5	1.53	....	0.45	0.47	0.47	1.06
1912.							
Jan.	— 6.7	0.50	....	0.05	0.05	0.05	0.45
Feb	10.6	0.23	....	0.27	0.26	0.26	—0.03
Mar.	20.8	0.48	....	0.47	0.49	0.49	—0.01
Apr	46.6	2.43	0.2	1.05	1.10	1.30	1.13
May	56.4	6.58	1.1	2.57	2.70	3.80	2.78
June	61.8	1.84	1.5	1.10	1.16	2.66	—0.82
July	68.9	3.47	1.7	1.95	2.05	3.75	—0.28
Aug.	63.8	4.89	1.7	2.12	2.23	3.93	0.96
Sept.	59.0	1.85	0.9	0.90	0.95	1.85	0.00
Oct.	48.1	0.93	0.1	0.38	0.40	0.50	0.43
	....	26.48	7.2	11.72	12.31	19.51	6.97

any given time. The soil storage capacity on any given water-shed at the time of break-up in the spring, for example, will depend, not only on the character of the soil, but on the quantity of water held over from the preceding fall, in the upper layers of soil, because the ground remained frozen through the winter. A portion of the snow which does not percolate into the ground, on melting, will immediately run off into the streams. Another portion will be retained for some time, part to appear as run-off later, and part to be gradually absorbed by the soil. The proportion retained for a time depends largely on topographic conditions. It may be treated substantially as precipi-

tation minus losses for the following month, resulting from well-distributed rains.

The curves of Fig. 31 are to be used to aid in determining the quantity of surface run-off resulting from a given monthly "precipitation minus losses". These curves must also be used in connection with daily precipitation records. In Minnesota, less surface run-off will, in general, result from a given "precipitation minus losses" in spring and fall than in summer, because the rains are usually well distributed during the former seasons. Some latitude must be allowed in the application of these curves.

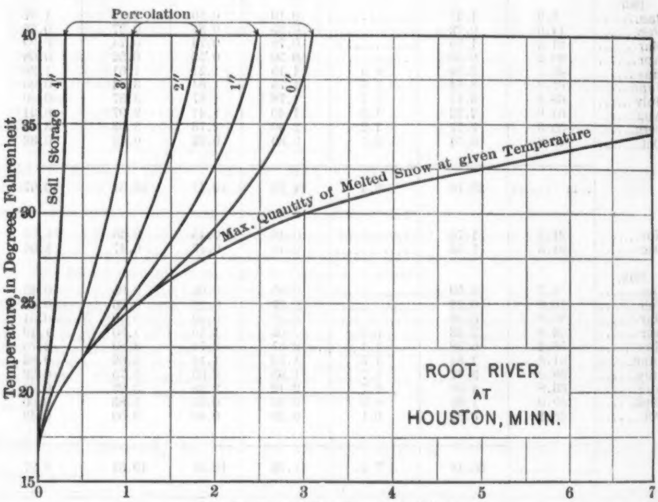


FIG. 30.

The curve of Fig. 32 is to be used to determine the seepage flow for a given quantity of subsoil storage. On the Root River water-shed, moisture which has once passed down through the upper foot or two of soil will continue downward, as a rule, to join the subsoil storage and aid in maintaining stream flow. It is practically safe against return through the action of capillarity.

The shape of the curves for other water-sheds of the type shown for the Root River water-shed varies mostly with topography, character of subsoil, and size of drainage basin.

In order to show the application of the writer's method of computing the monthly distribution of annual run-off, the detailed computations for the Root River water-shed at Houston, Minn., are given in Table 35. Summing up the monthly run-off figures, as computed in this way, gives a value for the annual run-off which is much more accurate, and differs somewhat, it will be noted, from the computed

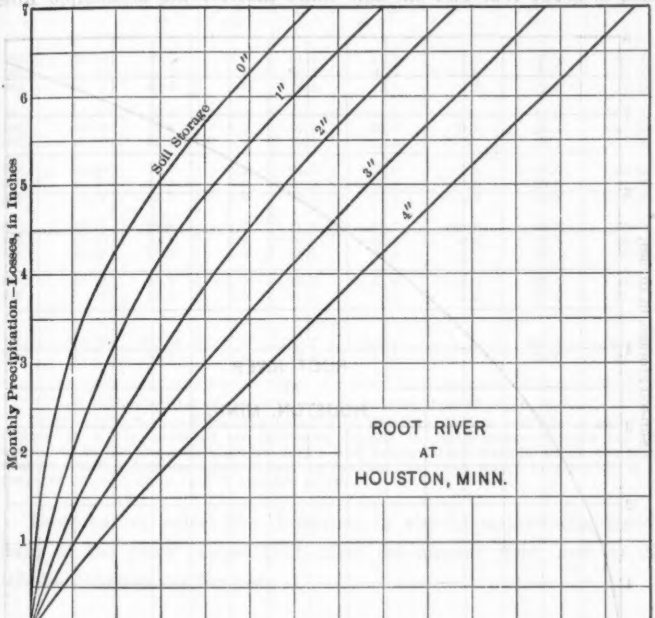


FIG. 31.

annual "precipitation minus losses" given in the summary of data and computations for the Root River water-shed. The difference results from surface run-off between November 1st and March 1st, and from differences in ground-water storage at the close of the water year. The average annual computed "precipitation minus losses" for the 4 years over which discharge records extend agrees substantially with the average annual observed run-off.

Frequency Curves.

The curves of Fig. 33 are presented in order to show the great difference in the annual run-off from the Mississippi River and the Tohickon Creek water-sheds, and its distribution.

The most striking facts, perhaps, are that the maximum rate of run-off, in inches of depth per month, for the Tohickon Creek water-shed is about four and one-half times that for the Mississippi River

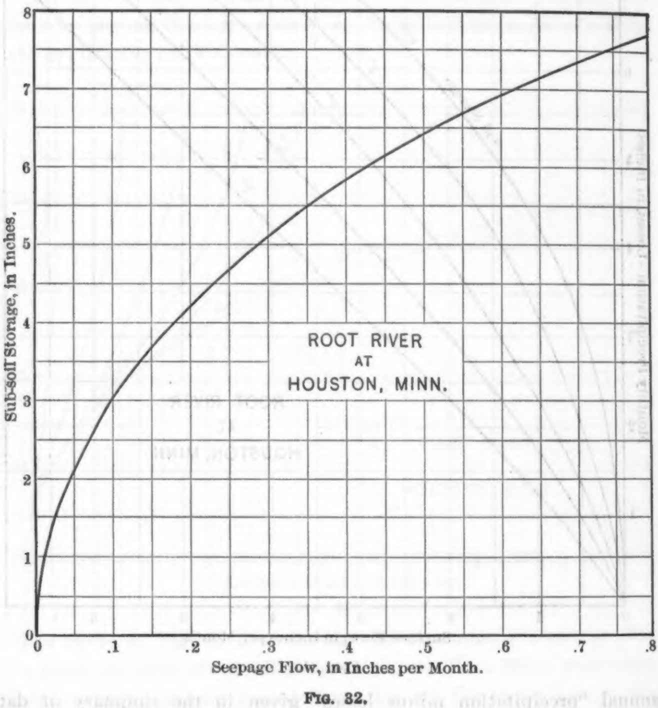


Fig. 32.

water-shed, and that the minimum rate of run-off from the former water-shed is about one-half the minimum rate from the latter. For about 10% of the time the rate of run-off from the Mississippi River water-shed is greater than that of the Tohickon Creek, in spite of the fact that the annual yield of the former water-shed is only about one-fifth of that of the latter.

TABLE 17.—SUMMARY OF DATA AND COMPUTATIONS FOR THE OHIO RIVER WATER-SHED, ABOVE WHEELING, W. VA.

Area = 23 820 sq. miles.

Year.	Rainfall.*	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	Change in storage.	Computed run-off.	Observed run-off.*
1892....	42.53	15.5	5.9	21.4	21.1	—0.3	21.4	22.01
1893....	41.95	14.75	5.8	20.55	21.4	—0.2	21.6	21.93
1894....	37.17	13.9	5.4	19.3	17.9	0.5	17.4	17.78
1895....	32.07	11.8	5.8	17.6	14.5	—0.4	14.9	16.14
1896....	45.79	16.8	6.1	22.9	22.9	0.5	22.4	21.24
1897....	36.60	13.5	5.7	19.2	17.4	—0.7	18.1	21.03
1898....	50.66	17.5	6.1	23.6	27.0	1.1	25.9	25.64
1899....	40.30	14.5	6.0	20.5	19.8	—0.8	20.6	21.93
1900....	37.02	13.7	5.9	19.6	17.4	0.3	17.1	17.60
1901....	43.15	15.8	5.7	21.5	21.7	—0.6	22.3	24.72
1902....	43.22	15.1	5.8	20.9	22.3	0.8	21.5	25.06
1903....	45.08	15.5	5.8	21.3	23.8	—0.1	23.9	27.08
1904....	40.73	13.9	5.9	19.8	20.9	—0.2	21.1	24.99
1905....	40.00	15.0	6.0	21.0	19.0	0.0	19.0	21.20
Total...	576.27	207.25	81.9	289.1	287.1	....	287.1	308.37
Mean ..	41.1	14.8	5.8	20.6	20.5	....	20.5	22.0

* From November 1st of previous year to October 31st of given year.

NOTE.—It is believed that the location of Weather Bureau stations in cities and villages, as a rule situated on railroads, results in temperature records for this water-shed which are somewhat too large, and precipitation records which are somewhat too small. Any such discrepancy in fundamental data would be reflected in the computed run-off, as in fact it appears to be.

These curves reflect the difference in size of water-sheds, besides many of the other factors that affect the annual yield, and its distribution throughout the year.

SEEPAGE FLOW AND SURFACE RUN-OFF.

The writer believes that the percentage which seepage flow constitutes of the total run-off on most streams of the Northwest is not fully realized. It is safe to say that, but for a few exceptional years, practically all the water in Minnesota streams from the first of November to the first of March is seepage flow. During the break-up and the spring rains following it, the ground-water storage which sustained the stream flow during the winter is again replenished. During July and August the demands of evaporation and transpiration consume practically all the available rainfall, so that the stream flow is again main-

TABLE 18.—DATA AND COMPUTATIONS FOR THE OHIO RIVER WATER-SHED.

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area.				Precipitation minus total loss, in inches.
			Transpiration, in inches.	Evaporation, in inches.		Total loss, in inches.	
				From curve.	Actual.		
1894.							
Nov......	40	2.54	....	0.70	0.61	0.61	1.93
Dec......	36	3.82	....	0.85	0.74	0.74	3.08
1895.							
Jan......	27	4.22	....	0.95	0.83	0.83	3.39
Feb......	24	1.19	....	0.60	0.52	0.52	0.67
Mar......	40	2.34	....	0.90	0.79	0.79	1.55
Apr......	53	2.68	0.2	1.20	1.05	1.25	1.43
May	62	2.18	0.8	1.20	1.05	1.85	0.33
June.....	73	3.12	1.3	1.90	1.66	2.96	0.16
July.....	70	3.38	1.4	1.80	1.57	2.97	0.41
Aug	72	3.41	1.3	1.85	1.62	2.92	0.49
Sept......	69	2.06	0.7	1.20	1.05	1.75	0.31
Oct......	48	1.13	0.1	0.40	0.35	0.45	0.68
	....	32.07	5.8	13.55	11.84	17.64	14.43
Nov......	40	3.55	....	0.90	0.79	0.79	2.76
Dec......	38	3.57	....	0.85	0.74	0.74	2.83
1896.							
Jan......	32	1.72	....	0.75	0.66	0.66	1.06
Feb......	33	3.21	....	1.00	0.88	0.88	2.33
Mar......	35	3.73	....	1.10	0.96	0.96	2.77
Apr......	58	2.70	0.2	1.40	1.23	1.43	1.27
May	69	2.97	0.8	1.75	1.53	2.33	0.64
June.....	70	5.23	1.4	2.70	2.36	3.76	1.47
July.....	73	9.08	1.6	4.25	3.72	5.32	3.76
Aug	72	2.83	1.3	1.60	1.40	2.70	0.13
Sept.....	64	4.85	0.7	2.10	1.84	2.54	2.31
Oct......	49	2.35	0.1	0.80	0.70	0.80	1.55
	....	45.79	6.1	19.20	16.81	22.91	22.88

tained by the ground-water supply. In September and October, fall rains, though largely absorbed, nevertheless usually supply a certain quantity of surface run-off. It is believed that on an average one-half to three-fourths of the annual run-off may be considered as being seepage flow.

On water-sheds such as that of the Colorado River in Texas, where the precipitation is distributed throughout the year in a similar manner to that prevailing throughout the Northwest, but where temperatures are so high that evaporation losses absorb by far the greater portion of the rainfall, and where the demands of vegetation for

moisture are never fully supplied, most of the stream flow results from surface run-off. On such water-sheds as that of the Colorado, practically all stream flow above a few hundredths of an inch of depth per month results from heavy rains commonly called "cloudbursts" over

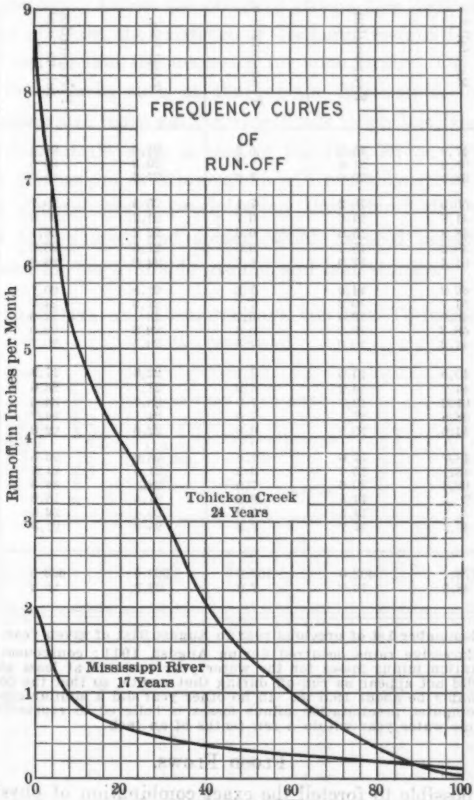


Fig. 33.

small areas. As the surface of the ground on such a water-shed is usually very dry, percolation is not very rapid, hence some of the rainfall escapes over the surface of the ground as run-off before sufficient time has elapsed for it to be evaporated, or to be used by vegetation.

TABLE 19.—SUMMARY OF DATA AND COMPUTATIONS FOR THE TOHICKON CREEK WATER-SHED, ABOVE POINT PLEASANT, PA.

Area = 102 sq. miles.

Year.	Rainfall.*	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	Observed run-off.*
1888	47.9	15.0	6.5	21.5	26.4	30.4
1889	65.8	21.8	7.5	29.3	36.5	40.3
1890	63.1	20.0	7.6	27.6	35.5	35.8
1891	55.0	18.2	7.1	25.3	29.7	30.6
1892	41.8	14.6	6.7	21.3	20.5	23.4
1893	48.3	15.6	6.6	22.2	26.1	27.7
1894	45.2	15.1	6.5	21.6	23.6	26.6
1895	51.4	17.9	6.7	24.6	26.8	29.5
1896	42.2	14.0	7.5	21.5	20.7	15.6
1897	51.7	19.3	7.7	27.0	24.7	23.5
1898	46.0	16.4	6.7	23.1	22.9	23.8
1899	48.3	15.7	6.6	22.3	26.0	31.8
1900	44.2	16.0	7.5	23.5	20.7	19.2
1901	40.5	15.5	7.1	22.6	17.9	17.1
1902	44.7	14.2	6.9	21.1	23.6	27.3
1903	61.2	19.7	7.0	26.7	34.5	37.6
1904	48.8	17.1	7.0	24.1	24.7	25.5
1905	44.6	15.8	6.8	22.6	22.0	22.8
1906	48.7	17.6	7.6	25.2	23.5	23.2
1907	43.1	15.1	6.5	21.6	21.5	22.0
1908	62.4	20.6	7.5	28.1	34.3	33.3
1909	39.6	14.6	6.8	21.4	18.2	17.0
1910	42.4	14.5	5.9	20.4	22.0	24.5
1911	46.5	17.7	7.3	25.0	21.5	17.4
Total..	1 173.4	402.0	167.6	569.6	603.8	625.9
Mean..	48.9	16.7	7.0	23.7	25.2	26.1

* From September 1st of previous year to August 31st of given year.

NOTE.—Excessive rains occurred during August, 1911; consequently, the computed precipitation minus losses for the water year contains at least about 2 in. of water which did not appear as run-off during that month, so that the computed run-off would probably be about 19.5 in. In no other year did a similar condition occur, so that the computed precipitation minus losses is believed to represent the run-off during the same water year within a few tenths of an inch.

FLOOD FLOWS.

It is impossible to foretell the exact combination of physical conditions which will result in the maximum flood in any given drainage basin. The maximum spring flood on such a water-shed as that of the Root River would occur at approximately such a time as the middle of March, 1913, when the temperature suddenly rose from relatively cold to considerably above freezing all day, and when several inches of warm rains, precipitated on frozen ground covered with the winter's

accumulation of snow, resulted in sending most of the surface storage plus the precipitation into the stream in a few days.

The conditions resulting in maximum floods on different water-sheds vary considerably, and each stream must be studied in detail as a separate problem. Unless the observed stream-flow data extend over a long period of years, the existence of discharge records for the given stream will not obviate the necessity for such study. On the larger Minnesota water-sheds, such as that of the Mississippi, local differences in temperature make early spring floods much less frequent than on smaller water-sheds, such as that of the Root River. The highest flood on the Mississippi at Minneapolis will probably occur in May or June, after general heavy precipitation throughout the water-shed, following a fall, winter, and spring which favored percolation, and thus exhausted all the available ground and lake storage.

TABLE 20.—DATA AND COMPUTATIONS FOR THE TOHICKON CREEK WATER-SHED.

Year and month.	Monthly temperature, in degrees Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area.			Total loss, in inches.	Precipitation minus total loss, in inches.	Recorded run-off.
			Transpiration, in inches.	Evaporation, in inches.				
				From curve.	Actual.			
1898.								
July	76.9	4.0	1.5	2.42	2.18	3.68	0.32	0.08
Aug	74.1	6.1	1.5	3.05	2.75	4.25	1.85	0.75
Sept	68.3	2.0	0.9	1.15	1.03	1.93	0.07	0.09
Oct	56.0	5.2	0.4	1.95	1.76	2.16	3.04	0.61
Nov	42.6	7.0	....	1.75	1.58	1.58	5.42	4.49
Dec	32.5	3.5	....	0.70	0.63	0.63	2.87	4.24
1899.								
Jan	28.2	3.7	....	0.92	0.83	0.83	2.87	4.75
Feb	25.2	4.8	....	0.90	0.81	0.81	3.99	5.63
Mar	38.2	6.6	....	1.70	1.53	1.53	5.07	9.01
Apr	50.1	2.2	0.3	1.00	0.90	1.20	1.00	1.58
May	60.6	2.2	0.9	1.20	1.08	1.98	0.22	0.25
June	71.1	2.7	1.3	1.65	1.48	2.78	−0.08	0.08
July	73.6	3.3	1.4	2.00	1.80	3.20	0.10	0.08

NOTE.—In order to get a better comparison between the computed and recorded run-off for the year 1898 to 1899, the computations are given for the 13 months beginning July, 1898, and ending July, 1899. It will be noted that the recorded run-off for July of both years is exactly the same, and very small, indicating equalization of ground storage between these dates, and thus affording a better comparison between the computed and recorded run-off. It is worthy of note that the rainfall from October to March, inclusive, was 30.8 in., and the recorded run-off was 28.73 in., or 93 per cent. Assuming the records to be correct, this would indicate an evaporation loss of less than 0.4 in. per month for temperatures ranging from 25 to 56 degrees. As the evaporation was unquestionably at least twice as much, there can be no doubt that either the rainfall or run-off records are decidedly in error.

TABLE 20.—(*Continued.*)

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area.			Total loss, in inches.	Precipitation minus total loss, in inches.
			Transpiration, in inches.	Evaporation, in inches.			
				From curve.	Actual.		
1900.							
Sept	69	1.6	0.9	0.95	0.86	1.76	—0.16
Oct	59	3.0	0.3	1.32	1.19	1.49	1.51
Nov	46	2.6	....	0.85	0.77	0.77	1.83
Dec	32	2.6	....	0.52	0.47	0.47	2.13
1901.							
Jan	30	2.6	....	0.85	0.77	0.77	1.83
Feb	23	0.9	....	0.52	0.47	0.47	0.43
Mar	39	4.8	....	1.40	1.26	1.26	3.54
Apr	49	5.7	0.3	2.05	1.85	2.15	3.55
May	59	5.2	1.1	2.30	2.07	3.17	2.03
June	69	1.8	1.4	1.20	1.08	2.48	—0.68
July	78	2.6	1.6	1.75	1.58	3.18	—0.58
Aug	74	7.1	1.5	3.45	3.10	4.60	2.50
	....	40.5	7.1	17.16	15.47	22.57	17.93
Sept	66	2.5	1.0	1.30	1.07	2.07	0.43
Oct	54	1.5	0.3	0.68	0.61	0.91	0.59
Nov	38	2.0	....	0.52	0.47	0.47	1.53
Dec	31	7.8	....	1.05	0.94	0.94	6.86
1902.							
Jan	28	3.1	....	0.85	0.76	0.76	2.34
Feb	26	5.6	....	0.95	0.85	0.85	4.75
Mar	43	4.2	....	1.40	1.26	1.26	2.94
Apr	50	3.9	0.3	1.56	1.40	1.70	2.20
May	60	1.6	0.9	0.95	0.85	1.75	—0.15
June	68	5.1	1.5	2.58	2.32	3.82	1.28
July	73	4.2	1.6	2.40	2.16	3.76	0.44
Aug	70	3.2	1.3	1.70	1.53	2.83	0.37
	....	44.7	6.9	15.94	14.22	21.12	23.58
Sept	64	7.6	1.0	3.05	2.75	3.75	3.85
Oct	55	5.8	0.3	2.05	1.85	2.15	3.65
Nov	49	1.7	....	0.62	0.56	0.56	1.14
Dec	29	7.6	....	0.92	0.83	0.83	6.77
1903.							
Jan	29	4.3	....	1.05	0.94	0.94	3.36
Feb	32	5.1	....	1.22	1.10	1.10	4.00
Mar	48	4.2	....	1.60	1.44	1.44	2.76
Apr	51	4.6	0.3	1.80	1.62	1.92	2.68
May	64	0.5	0.7	0.45	0.40	1.10	—0.60
June	65	8.6	1.7	3.60	3.24	4.94	3.66
July	73	6.4	1.7	3.25	2.93	4.63	1.77
Aug	69	4.8	1.3	2.30	2.07	3.37	1.43
	....	61.2	7.0	21.91	19.73	26.73	34.47

TABLE 20.—(*Continued.*)

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area. Transpiration, in inches.	Evaporation, in inches. From curve.	Evaporation, in inches. Actual.	Total loss, in inches.	Precipitation minus total loss, in inches.
1906.							
Sept........	68	1.7	0.8	1.00	0.90	1.70	0.00
Oct..........	55	6.7	0.4	2.30	2.07	2.47	4.23
Nov........	43	1.6	....	0.50	0.45	0.45	1.15
Dec..........	31	4.6	....	0.85	0.77	0.77	3.83
1907.							
Jan........	30	4.0	....	1.00	0.90	0.90	3.10
Feb........	23	2.8	....	0.70	0.63	0.63	2.17
Mar........	43	3.2	....	1.20	1.08	1.08	2.12
Apr..........	47	3.4	0.2	1.30	1.17	1.37	2.03
May........	57	3.5	0.8	1.65	1.48	2.28	1.22
June........	65	5.1	1.5	2.50	2.25	3.75	1.35
July........	74	3.0	1.5	1.90	1.71	3.21	−0.21
Aug........	70	3.5	1.3	1.85	1.66	2.96	0.54
	....	43.1	6.5	16.75	15.07	21.58	21.53

TABLE 21.—Summary of Data and Computations for the James River Water-Shed, Above Cartersville, Va.

Area = 6 230 sq. miles.

Year.	Rainfall.*	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	Change in storage.	Computed run-off.	Observed run-off.*
1899	44.5	16.4	7.1	23.5	21.00	−1.4	22.4	20.3
1900	37.7	15.3	6.5	21.8	15.90	0.6	15.3	14.0
1901	51.6	20.5	7.3	27.8	23.80	−1.0	24.8	24.0
1902	43.3	14.2	7.0	21.2	22.10	1.5	20.6	19.6
1903	46.9	17.5	7.1	24.6	22.30	−1.0	23.3	23.9
1904	28.7	12.6	6.7	19.3	9.40	−0.5	9.9	10.3
1905	41.7	17.3	7.6	24.9	16.30	0.6	16.2	14.1
Total.	294.4	113.8	49.3	163.1	131.30	....	132.5	126.2
Mean.	42.1	16.3	7.0	23.3	18.8	....	18.9	18.0

* From November 1st of previous year to October 31st of given year.

TABLE 22.—DATA AND COMPUTATIONS FOR THE JAMES RIVER WATER-SHED.

Year and month.	Monthly temperature, in degrees Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area.			Total loss, in inches.	Precipitation minus total loss, in inches.
			Transpiration, in inches.	Evaporation, in inches.			
				From curve.	Actual.		
1899.							
Nov	45.8	0.93	....	0.35	0.32	0.32	0.61
Dec	34.9	1.90	....	0.45	0.42	0.42	1.48
1900.							
Jan.....	34.6	3.35	....	1.01	0.94	0.94	2.41
Feb.....	32.7	4.41	....	1.16	1.07	1.07	3.34
Mar	40.8	3.76	....	1.25	1.16	1.16	2.60
Apr	54.1	2.84	0.4	1.31	1.21	1.61	1.23
May....	63.4	2.92	1.1	1.60	1.48	2.58	0.34
June...	69.9	5.34	1.5	2.72	2.52	4.02	1.32
July....	75.6	2.98	1.4	1.88	1.74	3.14	−0.16
Aug....	78.2	1.54	1.0	1.10	1.02	2.02	−0.48
Sept ...	71.2	3.98	0.8	2.05	1.90	2.70	1.28
Oct.. ...	60.3	3.75	0.3	1.62	1.50	1.80	1.95
	....	37.70	6.5	16.50	15.28	21.78	15.92
Nov.....	47.5	3.12	....	1.05	0.97	0.97	2.15
Dec.....	36.7	2.78	....	0.69	0.64	0.64	2.14
1901.							
Jan.....	35.3	2.67	....	0.93	0.86	0.86	1.81
Feb	31.4	0.59	....	0.50	0.46	0.46	0.13
Mar....	44.6	3.48	....	1.30	1.20	1.20	2.28
Apr	48.9	6.92	0.3	2.35	2.17	2.47	4.45
May....	61.9	6.70	1.1	2.90	2.68	3.78	2.92
June...	70.5	5.62	1.6	2.87	2.66	4.26	1.36
July....	77.0	5.23	1.7	2.99	2.77	4.47	0.76
Aug....	72.2	10.22	1.5	4.45	4.12	5.62	4.60
Sept....	64.8	3.75	0.8	1.80	1.67	2.47	1.28
Oct.....	54.0	0.55	0.3	0.30	0.28	0.58	−0.03
	....	51.63	7.3	22.13	20.48	27.78	23.85

The determination of the probable extremes of discharge for streams on which only short-term run-off records are available is too large a problem to be discussed in the present paper.

NEED OF SUPPLEMENTING OBSERVED STREAM-FLOW DATA.

On comparatively few streams of the country do the records of discharge extend over a long term of years. Short-term records do not give the extremes of high and low flow unless by sheer accident such

TABLE 23.—SUMMARY OF DATA AND COMPUTATIONS FOR THE ROANOKE RIVER WATER-SHED.

Area = 390 sq. miles.

Year.	Rainfall.*	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	Change in storage.	Computed run-off.	Observed run-off.*
1897......	38.1	15.4	6.8	22.2	15.9	0.8	15.1	15.22
1898......	40.7	17.2	7.0	24.2	16.5	1.2	15.3	14.44
1899......	44.1	16.7	7.0	23.7	20.4	—1.8	22.2	24.52
1900......	40.8	16.8	6.9	23.7	17.1	0.6	16.5	13.19
1901......	54.9	21.6	7.6	29.2	25.7	—0.8	26.5	29.82
1902......	38.0	13.8	6.2	20.0	18.0	0.5	17.5	18.17
1903......	47.5	18.1	7.0	25.1	22.4	—0.5	22.9	21.86
1904......	33.2	14.0	7.0	21.0	12.2	—0.7	12.9	8.54
1905......	45.8	18.3	7.5	25.8	20.0	1.0	19.0	14.65
Total....	383.1	151.9	63.0	214.9	168.2	....	167.9	159.41
Mean....	42.6	16.9	7.0	23.9	18.7	....	18.6	17.7

* From November 1st of previous year to October 31st of given year.

years have been included in the term over which observations extend. Short-term records, moreover, do not give a satisfactory value for mean utilizable flow. In the last analysis, it is usually necessary to supplement the observed stream-flow data with computed values based on rainfall and other physical data, in order to arrive at a probable maximum, minimum, and mean utilizable flow for any given stream.

In order to show the annual and periodic variations in rainfall and run-off on the two streams, considered in this paper, for which relatively long-term records are available, the curves in Figs. 34 to 36 have been prepared.

The curves of cumulative mean rainfall and run-off represent, at every point, the mean of all the annual values preceding. It is interesting to note that on the Mississippi River water-shed the 12-year mean run-off differs from the 6-year mean by more than 20 per cent. The 12-year mean rainfall, however, differs from the 6-year mean by only a small percentage. On the Tohickon Creek water-shed the 5-year mean rainfall is 111% of the 24-year mean, and the 5-year mean run-off is 123% of the long-term mean. Though the variation in run-off is proportionately very much larger than the variation in

TABLE 24.—DATA AND COMPUTATIONS FOR THE ROANOKE RIVER WATER-SHED.

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area.			Total loss, in inches.	Precipitation minus total loss, in inches.
			Transpiration, in inches.	Evaporation, in inches.			
				From curve.	Actual.		
1896.							
Nov.....	52.2	4.47	....	1.55	1.39	1.39	3.08
Dec.....	39.0	0.50	....	0.18	0.16	0.16	0.34
1897.							
Jan.....	33.6	1.64	....	0.40	0.36	0.36	1.28
Feb.....	40.6	7.10	....	1.99	1.79	1.79	5.31
Mar	50.6	3.74	....	1.55	1.39	1.39	2.35
Apr.....	55.6	1.67	0.4	0.92	0.83	1.23	0.44
May	63.0	3.71	1.0	1.90	1.71	2.71	1.00
June....	73.1	1.90	1.4	1.33	1.20	2.60	—0.70
July....	76.6	4.72	1.6	2.74	2.46	4.06	0.66
Aug.....	75.2	2.83	1.3	1.75	1.58	2.88	—0.05
Sept....	70.6	1.96	0.8	1.20	1.08	1.88	0.08
Oct.....	60.2	3.86	0.3	1.65	1.48	1.78	2.08
	....	38.10	6.8	17.16	15.43	22.23	15.87
Nov	48.6	1.13	....	0.46	0.42	0.42	0.71
Dec.....	40.6	3.72	....	1.00	0.90	0.90	2.82
1898.							
Jan.....	39.2	2.90	....	0.90	0.81	0.81	2.09
Feb.....	37.5	0.62	....	0.40	0.36	0.36	0.26
Mar	50.8	3.47	....	1.48	1.33	1.33	2.14
Apr.....	52.9	1.96	0.4	0.99	0.89	1.29	0.67
May	67.4	6.14	1.2	2.94	2.64	3.84	2.30
June	73.9	2.82	1.4	1.81	1.63	3.03	—0.21
July.....	77.4	4.32	1.6	2.58	2.32	3.92	0.40
Aug	77.2	3.59	1.3	2.11	1.90	3.20	0.39
Sept.....	71.2	3.75	0.8	2.00	1.80	2.60	1.15
Oct.....	58.8	6.32	0.3	2.40	2.16	2.46	3.86
	....	40.74	7.0	19.07	17.16	24.16	16.58

rainfall, the actual variation in inches of rainfall and run-off is practically the same for the Tohickon Creek water-shed, but the variation in inches of rainfall is very much greater than the variation in inches of run-off on the Mississippi River water-shed. This difference exists on all water-sheds having similar differences in annual rainfall and evaporation and transpiration losses. On the Tohickon Creek water-shed, the normal annual rainfall is sufficient to supply the needs of evaporation and transpiration; consequently,

TABLE 25.—SUMMARY OF DATA AND COMPUTATIONS FOR THE TOMBIGBEE RIVER WATER-SHED, ABOVE COLUMBUS, MISS.

Area = 4 440 sq. miles.

Year.	Rainfall.*	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	Change in storage.	Computed run-off.	Observed run-off.*
1901........	51.1	24.5	7.2	31.7	19.4	0.4	19.0	19.6
1902........	45.0	20.4	7.4	27.8	17.2	—0.4	17.6	17.3
1903........	50.5	21.6	7.1	28.7	21.8	—0.8	22.6	23.1
1904........	33.6	17.5	8.3	25.8	7.8	0.3	7.5	5.2
1905........	52.4	23.6	9.0	32.6	19.8	0.6	19.2	15.7‡
1906........	49.6	24.0	9.3	33.3	16.3	1.7†	14.6	12.8
1907........	50.3	23.6	9.5	33.1	17.2	2.2†	19.4	18.2
1908........	54.4	25.2	9.4	34.6	19.8	0.3	19.5	17.8
1909........	56.2	25.2	8.7	33.9	22.3	—0.3	22.6	24.1
Total......	443.1	205.6	75.9	281.5	161.6	....	162.0	153.8
Mean......	49.2	22.8	8.4	31.2	18.0	....	18.0	17.1

* From October 1st of previous year to September 30th of given year.

† 6.8 in. of rainfall in September, 1906. 1.5 in. assumed as held over into following year.

‡ 7 months estimated on basis of run-off at Epes.

speaking in very approximate terms, most of the rainfall in addition to those needs appears as run-off, as has been pointed out frequently in the past. On the Mississippi River water-shed, however, and throughout the greater part of the United States, the normal rainfall is insufficient to supply the needs of transpiration and evaporation at the prevailing temperatures; consequently, a large portion of any increased rainfall goes to supply unsatisfied needs of transpiration and evaporation, and hence a comparatively small portion of the increased rainfall, within certain limits, appears as run-off.

Sargent* comments briefly on long-term variations in stream flow on the Croton and Hudson Rivers. It appears from these records, in so far as low water is concerned, that the rate of flow for the 5 dryest months at Mechanicsville, on the Hudson River, was lowest in 1908 and highest in 1905. It was about one-third as much during the former year as during the latter. It also appears that the rate of flow which occurred 70% of the time during the 5 years, 1909 to 1913, was only

* *Engineering News*, December 3d, 1914.

a little more than one-half of that which occurred 70% of the time during the 26 years from 1888 to 1913, even though the extreme minimum rate of flow was practically the same in the two periods.

Fig. 36 shows the progressive 5-year mean rainfall and run-off for the Mississippi River and Tohickon Creek water-sheds. These curves bring out forcibly the great differences which exist, particularly in run-off, between the average values derived from short-term—that is, in this case, 5-year—records.

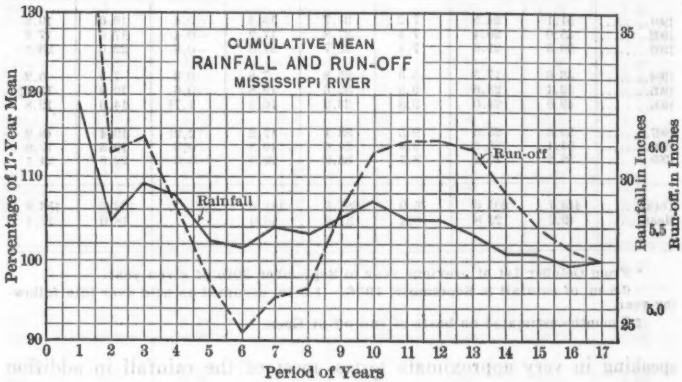


FIG. 34.

If the conclusion as to mean annual run-off for the Mississippi River water-shed were based on the 5-year mean ending in 1902, during which period the rainfall averaged 98% of the mean for the 17-year period, this conclusion would be 20% too low. If the conclusion were based on the 5-year mean ending in 1909, during which period the rainfall was 104% of the mean for the 17-year period, the figure would be 40% in error. If the conclusion were based on the 5-year period ending in 1913, during which period the rainfall was about 10% below normal, the value adopted would be nearly 35% too small. The maximum variation in 5-year means of run-off within the 17-year period over which the records used in this paper extend, is about 75 per cent.

Even though, on a small water-shed such as that of Tohickon Creek, the fluctuations are not as great as they are on the Mississippi, nevertheless, very substantial differences exist between the 5-year mean rainfall and run-off and the 24-year mean.

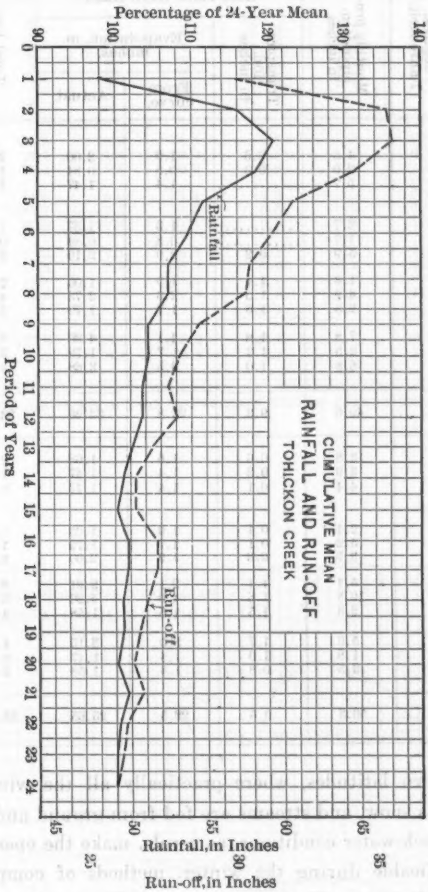

CUMULATIVE MEAN
RAINFALL AND RUN-OFF
TOHICKON CREEK
Percentage of 24-Year Mean
Period of Years
Rainfall
Run-off
Rainfall, in Inches
Run-off, in Inches

FIG. 35.

TABLE 26.—DATA AND COMPUTATIONS FOR THE TOMBIGBEE RIVER WATER-SHED.

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area.			Total loss, in inches.	Precipitation minus total loss, in inches.
			Transpiration, in inches.	Evaporation, in inches.			
				From curve.	Actual.		
1905.							
Oct.....	63	4.5	0.6	1.9	2.00	2.60	1.90
Nov....	53	2.0	0.3	0.8	0.84	1.14	0.86
Dec.....	40	5.7	....	1.4	1.47	1.47	4.23
1906.							
Jan.....	44	3.7	....	1.3	1.37	1.37	2.33
Feb.....	40	1.7	....	0.8	0.84	0.84	0.86
Mar....	47	5.9	0.2	2.0	2.10	2.30	3.60
Apr....	64	1.6	1.1	1.0	1.05	2.15	−0.55
May....	68	4.9	1.5	2.6	2.73	4.23	0.67
June...	78	2.5	1.6	1.7	1.79	3.39	−0.89
July....	78	7.8	1.8	4.1	4.30	6.10	1.70
Aug....	80	2.5	1.2	1.7	1.79	2.99	−0.49
Sept....	78	6.8	1.0	3.5	3.68	4.68	2.12
	....	49.6	9.3	22.8	23.96	33.26	16.34
Oct.....	60	3.8	0.6	1.6	1.68	2.28	1.52
Nov....	53	3.9	0.3	1.4	1.47	1.77	2.13
Dec.....	47	4.4	0.1	1.4	1.47	1.57	2.83
1907.							
Jan.....	51	2.1	0.1	1.0	1.05	1.15	0.95
Feb.....	45	5.2	0.1	1.7	1.79	1.89	3.31
Mar....	63	3.9	0.6	1.9	2.00	2.60	1.30
Apr....	55	5.1	1.1	2.1	2.21	3.31	1.79
May....	65	10.3	1.6	4.0	4.20	5.80	4.50
June...	75	2.3	1.5	1.5	1.58	3.08	−0.78
July....	81	5.0	1.7	3.0	3.15	4.85	0.15
Aug....	82	1.8	1.0	1.4	1.47	2.47	−0.67
Sept....	74	2.5	0.8	1.5	1.58	2.38	0.12
	....	50.3	9.5	22.5	23.65	33.15	17.15

In northern latitudes, where practically all the winter precipitation occurs as snow, and streams are fed from ground and lake storage, and where back-water conditions, as a rule, make the open-water rating curve inapplicable during the winter, methods of computing run-off also find application. Their principal use, however, lies in extending observed run-off records so as to include more nearly the extremes of discharge, and to give a more accurate value for mean utilizable flow, than can possibly be obtained from short-term records.

TABLE 27.—SUMMARY OF DATA AND COMPUTATIONS FOR THE COLORADO RIVER WATER-SHED, ABOVE AUSTIN, TEX.

Area = 37 000 sq. miles.

Year.	Rainfall.*	Evaporation.	Transpiration	Total loss.	Precipitation minus total loss.	Change in storage.	Computed run-off.	Observed run-off.*
1900	37.4	23.9	10.9	34.8	2.6	0	2.6	0.62
1901	15.7	10.7	4.3	15.0	0.7	0	0.7	0.95
1902	34.0	21.4	8.8	30.2	3.8	2.5	1.3	0.62
1903	21.1	14.8	8.6	23.4	−2.3	−2.5	0.2	0.75
1904	24.4	16.1	7.5	23.6	0.8	0.5	0.3	0.70
1905	31.7	21.1	10.0	31.1	0.6	−0.8	1.4	1.14
1906	25.6	18.3	7.1	25.4	0.2	0	0.2	0.68
1907	28.1	17.3	7.9	25.2	2.9	0.4	2.5	0.98
1908	29.2	19.1	10.1	29.2	0.0	−0.2	0.2	0.62
1909	21.6	14.6	5.7	20.3	1.3	0.1	1.2	0.30
Total.	268.8	177.3	80.9	258.2	10.6	....	10.6	7.36
Mean.	26.9	17.7	8.1	25.8	1.1	....	1.06	0.74

*From March 1st of given year to February 28th or 29th of following year.

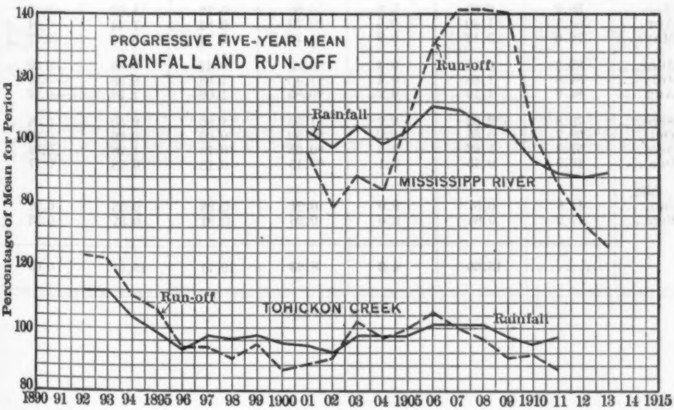


FIG. 36.

TABLE 28.—DATA AND COMPUTATIONS FOR THE COLORADO RIVER WATER-SHED.

Year and month.	Monthly temperature, in degrees Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area. Transpiration, in inches.	Loss from Land Area. Evaporation, in inches. From curve.	Loss from Land Area. Evaporation, in inches. Actual.	Total loss, in inches.	Precipitation minus total loss, in inches.
1900.							
Mar.....	56.9	3.38	0.4	1.60	1.92	2.32	1.06
Apr.....	64.8	7.63	1.2	3.35	4.02	5.22	2.41
May.....	72.4	5.47	1.7	2.85	3.42	5.12	0.35
June....	82.0	0.42	1.3	0.45	0.54	1.84	—1.42
July.....	80.6	3.97	1.5	2.51	3.02	4.52	—0.55
Aug	79.8	3.12	1.2	2.00	2.40	3.60	—0.48
Sept....	79.3	6.40	1.6	3.45	4.13	5.73	0.67
Oct......	69.0	3.17	1.0	1.75	2.10	3.10	0.07
Nov.....	57.3	1.10	0.5	0.55	0.66	1.16	—0.06
Dec......	48.4	1.07	0.2	0.41	0.49	0.69	0.38
1901.							
Jan.....	50.8	0.25	0.2	0.31	0.37	0.57	—0.32
Feb	46.0	1.45	0.1	0.71	0.85	0.95	0.50
	....	37.43	10.9	19.94	23.92	34.82	2.61
Mar.....	56.2	0.90	0.3	0.60	0.72	1.02	—0.12
Apr.....	62.1	1.40	0.6	*0.70	0.84	1.44	—0.04
May.....	72.6	3.18	0.9	*1.76	2.11	3.01	0.17
June....	82.3	0.52	0.4	*0.35	0.42	0.82	—0.30
July....	84.8	1.88	0.4	*1.40	1.68	2.08	—0.20
Aug....	84.8	1.10	0.2	*0.85	1.02	1.22	—0.12
Sept....	76.8	2.52	0.7	*1.50	1.80	2.50	0.02
Oct.....	69.7	0.88	0.2	*0.50	0.60	0.80	0.08
Nov.....	57.9	1.62	0.5	*0.65	0.78	1.28	0.34
Dec.....	46.9	0.45	0.1	*0.15	0.18	0.28	0.17
1902.							
Jan.....	44.8	0.57	0.0	0.21	0.25	0.25	0.32
Feb.....	47.2	0.64	0.0	0.26	0.31	0.31	0.33
	....	15.66	4.8	8.93	10.71	15.01	0.65

* Low storage.

TABLE 29.—SUMMARY OF DATA AND COMPUTATIONS FOR THE SACRAMENTO RIVER WATER-SHED, ABOVE RED BLUFF, CAL.

Area = 10 400 sq. miles.

Year.	Rainfall.*	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	Observed run-off.*
1903	28.3	8.3	2.2	10.5	17.8	17.60
1904	38.9	9.1	2.0	11.1	27.8	28.34
1905	37.8	12.0	3.7	15.7	22.1	19.23
1906	33.5	9.5	3.0	12.5	21.0	19.92
1907	35.7	8.8	1.7	10.5	25.2	24.67
1908	25.3	7.5	1.8	9.3	16.0	14.04
1909	37.5	7.3	1.7	9.0	28.5	25.82
1910	23.1	6.9	2.3	9.2	13.9	16.14
1911	29.6	7.2	3.0	10.2	19.4	17.84
Total	289.7	76.6	21.4	98.0	191.8	183.60
Mean	32.2	8.5	2.4	10.9	21.3	20.4

* September 1st of previous year to August 31st of given year.

NOTE.—A water year from September 1st to August 31st was used for this water-shed in order to permit the highest possible equalization of storage. September of practically every year marks a decided increase in precipitation after several months of distinctly dry weather.

TABLE 30.—DATA AND COMPUTATIONS FOR THE SACRAMENTO RIVER WATER-SHED.

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area.			Total loss, in inches.	Precipitation minus total loss, in inches.
			Transpiration, in inches.	Evaporation, in inches.			
				From curve.	Actual.		
1903.							
Sept	61	0.2	0.1	0.20	0.17	0.27	—0.07
Oct	55	1.4	0.3	0.65	0.55	0.85	0.55
Nov	44	8.0	0.2	1.95	1.66	1.86	6.14
Dec	38	2.5	....	0.65	0.55	0.55	1.95
1904.							
Jan	35	2.0	....	0.80	0.68	0.68	1.32
Feb	40	9.0	....	1.97	1.68	1.68	7.32
Mar	38	11.0	....	1.80	1.53	1.53	9.47
Apr	48	3.8	0.4	1.48	1.26	1.66	2.14
May	58	0.6	0.5	0.50	0.42	0.92	—0.32
June	66	0.1	0.3	0.27	0.23	0.53	—0.43
July	73	0.3	0.2	0.37	0.31	0.51	—0.21
Aug	74	0.0	0.0	0.11	0.09	0.09	—0.09
	....	38.9	2.0	10.75	9.13	11.13	27.79
Sept	66	3.2	0.1	1.63	1.39	1.49	1.71
Oct	55	5.5	0.3	1.96	1.67	1.97	3.53
Nov	48	4.3	0.1	1.35	1.15	1.25	3.05
Dec	37	3.7	....	0.89	0.76	0.76	2.94
1905.							
Jan	41	6.0	0.1	1.75	1.49	1.59	4.41
Feb	44	3.0	0.3	1.15	0.98	1.28	1.72
Mar	47	7.0	0.5	2.29	1.95	2.45	4.55
Apr	54	2.0	0.8	1.00	0.85	1.65	0.35
May	55	2.5	0.9	1.25	1.07	1.97	0.53
June	67	0.6	0.5	0.50	0.42	0.92	—0.32
July	77	0.0	0.1	0.19	0.16	0.26	—0.26
Aug	75	0.0	....	0.11	0.09	0.09	—0.09
	....	37.8	3.7	14.07	11.98	15.68	22.12

TABLE 31.—SUMMARY OF DATA AND COMPUTATIONS FOR THE PIT RIVER WATER-SHED, ABOVE BIEBER, CAL.

Area = 2 950 sq. miles.

Year.	Rainfall.*	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	Observed run-off.*
1904	19.4	8.2	3.2	11.4	8.0	....
1905	11.7	6.2	3.3	9.5	2.2	2.00
1906	15.1	6.8	3.0	9.8	5.3	4.78
1907	17.4	7.6	3.1	10.7	6.7	7.70
1908	9.8	5.9	2.6	8.5	1.3	1.23
1909	15.2	6.7	2.6	9.3	5.9	....
Total	88.6	41.4	17.8	59.2	29.4	15.66
Mean	14.8	6.9	3.0	9.9	4.9 †(3.87)	3.92

*From September 1st of previous year to August 31st of given year.
†Mean of years over which observations extend.

TABLE 32.—DATA AND COMPUTATIONS FOR THE PIT RIVER WATER-SHED.

Year and month.	Monthly temperature, in degrees Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area. Transpiration, in inches.	Loss from Land Area. Evaporation, in inches. From curve.	Loss from Land Area. Evaporation, in inches. Actual.	Total loss, in inches.	Precipitation minus total loss, in inches.
1906.							
Sept....	60	0.58	0.2	0.32	0.35	0.55	0.03
Oct.....	49	0.18	0.1	0.14	0.15	0.25	—0.07
Nov.....	36	1.24	....	0.30	0.33	0.33	0.91
Dec	34	2.72	....	0.60	0.66	0.66	2.06
1907.							
Jan.....	25	1.68	....	0.65	0.71	0.71	0.97
Feb.....	42	3.25	0.1	1.18	1.30	1.40	1.85
Mar	35	3.72	0.1	1.11	1.22	1.32	2.40
Apr	48	0.41	0.4	0.42	0.46	0.86	—0.45
May.....	52	1.26	0.8	0.70	0.77	1.57	—0.31
June....	55	1.84	0.9	0.95	1.05	1.95	—0.11
July....	67	0.28	0.3	0.35	0.38	0.68	—0.40
Aug.....	63	0.23	0.2	0.21	0.23	0.43	—0.20
	....	17.39	3.1	6.93	7.61	10.71	6.68
Sept....	57	1.10	0.3	0.54	0.59	0.89	0.21
Oct.....	54	1.62	0.4	0.75	0.82	1.22	0.40
Nov	38	0.91	....	0.26	0.29	0.29	0.62
Dec.....	35	1.90	....	0.44	0.48	0.48	1.42
1908.							
Jan.....	34	1.05		0.62	0.68	0.68	0.37
Feb.....	35	0.34	....	0.45	0.49	0.49	—0.15
Mar	37	0.40	....	0.45	0.49	0.49	—0.09
Apr	48	0.23	0.1	0.30	0.33	0.43	—0.20
May....	47	1.36	0.5	0.70	0.77	1.27	0.09
June...	57	0.64	0.7	0.48	0.53	1.23	—0.59
July....	72	0.16	0.4	0.25	0.27	0.67	—0.51
Aug....	67	0.10	0.2	0.15	0.17	0.37	—0.27
	....	9.81	2.6	5.39	5.91	8.51	1.30

TABLE 33.—SUMMARY OF DATA AND COMPUTATIONS FOR THE McCLOUD RIVER WATER-SHED, ABOVE GREGORY, CAL.

Area = 608 sq. miles.

Year.	Rainfall.*	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	Observed run-off.*
1903	53.2	6.9	1.7	8.6	44.6	47.80
1904	81.9	8.2	2.2	10.4	71.5	73.98
1905	62.5	10.6	2.6	13.2	49.3	52.10
1906	60.6	9.1	2.6	11.7	48.9	51.27
1907	66.0	7.8	2.7	10.5	55.5	59.87
1908	47.4	6.8	2.4	9.2	38.2	39.05
Total	371.6	49.4	14.2	63.6	308.0	324.07
Mean	61.9	8.2	2.4	10.6	51.3	54.0

* From September 1st of previous year to August 31st of given year.

NOTE.—A water year from September 1st to August 31st was used for this water-shed in order to permit the highest possible equalization of storage. September of practically every year marks a decided increase in precipitation after several months of markedly dry weather.

TABLE 34.—DATA AND COMPUTATIONS FOR THE McCLOUD RIVER WATER-SHED.

Year and month.	Monthly temperature, in degrees Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area. Transpiration, in inches.	Evaporation, in inches. From curve.	Evaporation, in inches. Actual.	Total loss, in inches.	Precipitation minus total loss, in inches.
1903.							
Sept....	62	0	0.1	0.10	0.06	0.16	–0.16
Oct.....	58	4.0	0.3	1.60	0.96	1.26	2.74
Nov.....	47	14.0	0.2	2.20	1.32	1.52	12.48
Dec.....	42	4.0	0.1	1.10	0.66	0.76	3.24
1904.							
Jan.....	39	4.5	....	1.10	0.66	0.66	3.84
Feb.....	42	20.0	....	2.20	1.32	1.32	18.68
Mar.....	40	28.0	....	2.00	1.20	1.20	26.80
Apr.....	50	7.0	0.4	2.40	1.44	1.84	5.16
May....	60	0.4	0.4	0.40	0.24	0.64	–0.24
June....	68	0	0.3	0.20	0.12	0.42	–0.42
July....	73	0	0.2	0.20	0.12	0.32	–0.32
Aug.....	74	0	0.2	0.10	0.06	0.26	–0.26
	....	81.9	2.2	13.60	8.16	10.36	71.54
Sept....	67	3.0	0.2	1.60	0.96	1.16	1.84
Oct.....	56	10.0	0.2	2.10	1.26	1.46	8.54
Nov.....	50	4.0	0.2	1.30	0.78	0.98	3.02
Dec.....	40	6.0		1.40	0.84	0.84	5.16
1905.							
Jan.....	43	12.0		2.20	1.32	1.32	10.68
Feb.....	45	7.0	0.1	2.20	1.32	1.42	5.58
Mar.....	48	12.0	0.2	2.70	1.62	1.82	10.18
Apr.....	53	4.0	0.4	1.70	1.02	1.42	2.58
May....	54	4.0	0.5	1.70	1.02	1.52	2.48
June....	65	0.5	0.5	0.40	0.24	0.74	–0.24
July....	76	0	0.2	0.20	0.12	0.32	–0.32
Aug.....	73	0	0.1	0.10	0.06	0.16	–0.16
	....	62.5	2.6	17.60	10.56	13.16	49.34

TABLE 35.—DATA AND COMPUTATIONS FOR THE ROOT RIVER WATER-SHED.

Year and month.	Monthly temperature, in degrees Fahrenheit.	Monthly precipitation, in inches.	Transpiration, in inches.	Evaporation, in Inches.		Total loss, in inches.	Precipitation minus total loss, in inches.	Computed Run-off, in Inches, over Water-shed.			Observed Run-off.		Storage over Water-shed, in Inches.					Notes.
				From curve.	Actual.			Surface flow.	Seepage flow.	Total.	At Houston.	At Lanesboro.	Monthly storage.	Cumulative storage on first of month.				
														Surface.	Soil.	Subsoil.	Total.	
1909.																		
June ...	67	3.6	2.0	1.90	2.33	4.33	−0.73	....	0.45	0.45	0.45	......	−1.18	......	2.00	6.10	8.10	
July....	71	0.7	1.3	0.60	0.74	2.04	−1.34	....	0.38	0.38	0.33	......	−1.72	......	1.30	5.62	6.92	
Aug....	74	7.9	2.4	3.75	4.60	7.00	0.90	0.30	0.32	0.62	0.69	......	0.28	......	0.00	5.20	5.20	(1)
Sept....	60	4.5	1.6	1.85	2.27	3.87	0.63	0.10	0.28	0.38	0.37	......	0.25	......	0.60	4.88	5.48	(2)
Oct.....	47	1.3	0.3	0.50	0.61	0.91	0.39	0.10	0.25	0.35	0.32	......	0.04	......	1.10	4.63	5.73	
Nov.....	39	6.5	....	1.50	1.84	1.84	4.66	0.80	0.21	1.01	0.83	......	3.65	......	1.50	4.27	5.77	(3)
Dec.....	15	1.5	....	0.30	0.37	0.37	1.13	0.20	0.22	0.42	0.44	......	0.71	2.00	3.00	4.42	9.42	(4)
1910.																		
Jan.....	13	1.7	....	0.30	0.37	0.37	1.33	....	....	0.38	0.37	......	0.95	2.50	2.00	5.63	10.13	
Feb.....	12	0.3	....	0.30	0.37	0.37	−0.07	....	....	0.33	0.30	0.24	−0.40	3.80	2.00	5.28	11.08	
										4.32	4.10							
Mar	45	0.0	0.2	0.30	0.25	0.45	−0.45	0.70	0.29	0.99	0.83	1.48	−1.44	3.70	2.00	4.98	10.68	(5)
Apr	51	1.2	0.8	0.60	0.74	1.54	−0.34	0.10	0.40	0.50	0.40	1.04	−0.84	0.50	3.00	5.74	9.24	
May....	53	2.6	1.3	1.10	1.35	2.65	−0.05	....	....	0.42	0.37	0.38	−0.47	......	2.50	5.90	8.40	
June ...	70	0.2	1.5	0.20	0.25	1.75	−1.55	....	....	0.36	0.28	0.28	−1.91	......	2.50	5.53	8.03	
July....	75	0.9	1.0	0.65	0.80	1.80	−0.90	....	....	0.31	0.24	0.24	−1.21	......	1.00	5.12	6.12	
Aug	70	4.1	1.4	2.10	2.57	3.97	0.13	....	....	0.27	0.25	0.32	−0.14	......	0.10	4.81	4.91	(6)
Sept....	60	3.5	1.2	1.50	1.84	3.04	0.46	0.05	0.24	0.29	0.33	0.40	0.17	......	0.20	4.57	4.77	
Oct.....	53	0.7	0.3	0.35	0.43	0.73	−0.03	....	0.22	0.22	0.25	0.26	−0.25	......	0.60	4.34	4.94	
Nov	29	0.4	....	0.10	0.12	0.12	0.28	0.05	0.20	0.25	0.25	0.29	0.03	......	0.50	4.19	4.69	
Dec.....	19	0.5	....	0.25	0.31	0.31	0.19	....	0.18	0.18	0.23	......	0.01	......	0.70	4.02	4.72	
1911.																		
Jan.....	13	0.9	....	0.30	0.37	0.37	0.53	0.05	0.16	0.21	0.23	......	0.32	0.20	0.70	3.83	4.73	(7)
Feb	23	1.5	....	0.60	0.74	0.74	0.76	0.30	0.15	0.45	0.56	*0.20	0.31	0.70	0.70	3.65	5.05	(8)
										4.45	4.22							

(1) 6½ in. of rain in 5 days.
(2) 4 in. of rain in 3 days.
(3) About 2 in. of snow; last week quite warm.
(4) Some snow melted during first week of month.
(5) High temperature, latter part of month.
(6) Heavy rain Aug. 31st and Sept. 5th.
(7) A few warm days, 24th-25th.
(8) 12th-17th warm, about 1 in. of rain.

TABLE 35.—(*Continued.*)

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Transpiration, in inches.	Evaporation, in Inches. From curve.	Evaporation, in Inches. Actual.	Total loss, in inches.	Precipitation minus total loss, in inches.	Computed Run-Off, in Inches, over Water-Shed. Surface flow.	Computed Run-Off. Seepage flow.	Computed Run-Off. Total.	Observed Run-Off. At Houston.	Observed Run-Off. At Lanesboro.	Storage over Water-Shed, in Inches. Monthly storage.	Cumulative storage on first of month. Surface.	Cumulative storage. Soil.	Cumulative storage. Subsoil.	Cumulative storage. Total.	Notes.
1911.																		
Mar....	35	0.8	....	0.40	0.49	0.61	0.31	0.15	0.14	0.29	0.29	0.24	0.02	0.50	1.30	3.56	5.36	(9)
Apr....	45	1.4	0.3	0.70	0.86	1.16	0.24	0.05	0.17	0.22	0.25	0.22	0.02	......	1.50	3.88	5.38	(10)
May....	64	5.4	1.6	2.50	3.06	4.66	0.74	0.15	0.15	0.30	0.32	0.33	0.44	......	1.70	3.70	5.40	(11)
June...	73	3.2	1.7	1.95	2.39	4.09	—0.89	0.10	0.14	0.24	0.27	0.21	—1.13	......	2.30	3.54	5.84	
July....	72	3.2	1.7	1.90	2.33	4.03	—0.83	....	0.12	0.12	0.21	0.11	—0.95	......	1.40	3.31	4.71	
Aug....	68	6.0	2.1	*2.60	3.18	5.28	0.72	0.20	0.11	0.31	0.81	1.76	0.41	......	0.60	3.16	3.76	(12)
Sept....	61	4.3	1.5	1.75	2.14	3.64	0.66	0.15	0.10	0.25	0.34	0.31	0.41	......	1.00	3.17	4.17	(13)
Oct.....	46	7.4	0.3	2.00	2.45	2.75	4.65	1.40	0.10	1.50	1.03	1.78	3.15	......	1.50	3.08	4.58	(14)
Nov....	25	1.9	....	0.40	0.49	0.49	1.41	0.30	0.20	0.50	0.38	0.47	0.91	......	3.50	4.23	7.73	(15)
Dec.....	25	2.8	....	0.50	0.61	0.61	2.19	0.60	0.29	0.89	0.65	0.82	1.30	0.70	3.00	4.94	8.64	(16)
1912.																		
Jan.....	—3	0.7	....	0.05	0.06	0.06	0.64	....	0.35	0.35	0.36	0.32	0.29	1.50	3.00	5.44	9.94	
Feb.....	14	0.4	....	0.30	0.37	0.37	0.03	....	0.31	0.31	0.28	0.25	—0.28	2.10	3.00	5.13	10.23	
										5.28	5.19							
Mar....	23	0.9	....	0.50	0.61	0.61	0.29	0.60	0.27	0.87	0.92	1.46	—0.58	2.10	3.00	4.85	9.95	(17)
Apr.....	49	2.9	0.4	1.20	1.47	1.87	1.03	0.80	0.34	1.14	1.12	1.54	—0.11	1.00	3.00	5.37	9.37	(18)
May.....	59	4.9	1.5	2.15	2.64	4.14	0.76	0.40	0.48	0.88	0.78	1.07	—0.12	......	3.00	6.26	9.26	(19)
June...	64	2.0	1.8	1.20	1.47	3.27	—1.27	....	0.45	0.45	0.40	0.40	—1.72	......	3.00	6.14	9.14	
July....	71	6.0	2.2	3.00	3.68	5.88	0.12	0.05	0.39	0.44	0.61	0.52	—0.32	......	1.70	5.72	7.42	(20)
Aug.....	67	3.5	2.0	1.70	2.08	4.08	—0.58	....	0.34	0.34	0.33	0.30	—0.92	......	1.70	5.40	7.10	(21)
Sept....	61	2.0	1.2	1.00	1.22	2.42	—0.42	....	0.30	0.30	0.30	0.28	—0.72	......	1.10	5.08	6.18	(22)
Oct.....	50	1.9	0.4	0.70	0.86	1.26	0.64	0.10	0.26	0.36	0.31	0.29	0.28	......	0.70	4.76	5.46	
Nov....	37	1.4	....	0.35	0.43	0.43	0.97	0.10	0.24	0.34	0.29	0.28	0.63	......	1.20	4.54	5.74	
Dec.....	25	1.7	....	0.40	0.49	0.49	1.21	0.10	0.22	0.32	0.24	......	0.89	......	2.00	4.37	6.37	(23)
1913.																		
Jan.....	16	0.5	....	0.40	0.49	0.49	0.01	....	0.20	0.20	0.20	0.18	—0.19	0.50	2.60	4.16	7.26	
Feb.....	14	0.8	....	0.35	0.43	0.43	0.37	....	0.18	0.18	0.18	0.17	0.19	0.50	2.60	3.97	7.07	
										5.82	5.68							

* Most of rain fell in one day—evaporation reduced about 0.1 in.
(9) Maximum temperature on one day, 76 degrees.
(10) Last days of March warm, some run-off carried forward into April; rain well distributed during April.
(11) About ½ in. of rain, May 31st and about 3 in. of rain, June 2d-4th.
(12) Most of rain in one day.
(13) Rain well distributed.
(14) Rain well distributed.
(15) Month cold; precipitation very well distributed.
(16) About 1 in. of rain on 10th.
(17) Month cold, except last week, which was quite warm.
(18) Rain well distributed.
(19) Rain very well distributed.
(20) Considerable rain on 9th-11th.
(21) Rain very well distributed.
(22) Rain very well distributed.
(23) About ½ in. of rain.

TABLE 35.—(*Continued.*)

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Transpiration, in inches.	Evaporation, in Inches.		Total loss, in inches.	Precipitation minus total loss, in inches	Computed Run-Off, in Inches, over Water-Shed.			Observed Run-Off.		Storage over Water-Shed, in Inches.					Notes.
				From curve.	Actual.			Surface flow.	Seepage flow.	Total.	At Houston.	At Lanesboro.	Monthly storage.	Cumulative storage on first of month.				
														Surface.	Soil.	Subsoil.	Total.	
1913.																		
Mar	27	3.2	....	0.90	1.10	1.10	2.10	1.50	0.16	1.66	*2.48	2.27	0.44	0.90	2.60	3.76	7.26	(24)
Apr......	46	1.5	0.3	0.75	0.92	1.22	0.28	0.20	0.26	0.46	0.46	......	—0.18	......	3.00	4.70	7.70	......
May	56	5.4	1.3	2.25	2.75	4.05	1.35	0.25	0.30	0.55	0.38	......	0.80	......	2.50	5.02	7.52	(25)
June	72	3.2	2.0	1.90	2.33	4.33	—1.13	....	0.34	0.34	0.28	......	—1.47	......	3.00	5.32	8.32	......
July	72	5.3	2.1	2.80	3.43	5.53	—0.23	0.10	0.30	0.40	0.42	......	—0.63	......	1.80	5.05	6.85	(26)
Aug.....	73	2.6	1.7	1.50	1.84	3.54	—0.94	....	0.27	0.27	0.30	......	—1.21	......	1.40	4.82	6.22	......
Sept	62	3.7	1.4	1.65	2.02	3.42	0.28	....	0.25	0.25	0.24	......	0.03	......	0.40	4.61	5.01	(27)
Oct......	47	2.6	0.3	0.85	1.05	1.35	1.25	0.10	0.20	0.30	0.28	......	0.95	......	0.70	4.24	4.94	(28)
Nov	41	1.7	....	0.50	0.61	0.61	1.09	0.10	0.19	0.29	0.25	......	0.80	......	1.80	4.09	5.89	(29)
Dec	31	0.2	....	0.08	0.10	0.10	0.10	....	0.20	0.20	0.26	......	—0.10	......	2.50	4.19	6.69	......
1914.																		
Jan	24	1.4	....	0.50	0.61	0.61	0.79	....	0.19	0.19	0.22	......	0.60	......	2.50	4.09	6.59	......
Feb......	11	0.5	....	0.20	0.20	0.24	0.26	....	0.17	0.17	0.17	......	0.09	0.80	2.50	3.89	7.19	......
										5.08	5.74							

(24) The month of destructive storms, minimum temperature, 10th–15th above freezing—heavy rain on 14th.
(25) Precipitation well distributed—no heavy downpours.
(26) Rather heavy rains 4th–5th.
(27) Rain well distributed.
(28) Rain well distributed.
(29) No snow—rain well distributed.
* Estimated from Lanesboro.

NOTE.—The 1913 run-off figures are advance estimates furnished through the courtesy of the District Office, U. S. G. S., and are subject to revision for publication. Official data will probably be available very soon.

DISCUSSION

Mr. Justin.

JOEL D. JUSTIN,* ASSOC. M. AM. SOC. C. E. (by letter).—The author has shown a keen appreciation of most of the factors which cause the relations between rainfall and run-off to vary from one water-shed to another. In the writer's opinion, however, he neglects to a large extent one of the most important factors. In his paper, "Derivation of Run-Off from Rainfall Data",† the writer found that slope and mean annual temperature (in the Eastern United States at any rate) were the chief factors which cause these relations to vary from one water-shed to another. It would seem to be almost self-evident that, rainfall and other factors being the same, the steeper water-shed will have the greater run-off and smaller evaporation.

The difficulties of making predictions as to the monthly distribution of run-off are very properly pointed out, and the necessity of taking account of the monthly temperature is emphasized by the author.

The writer agrees that it is not necessary to be able to estimate closely what the run-off was for any particular month of a certain year. It is sufficient, for all practical purposes, to decide on a distribution of run-off throughout the year, which, for the existing conditions of rainfall, temperature, and ground-water, might have taken place. The writer has shown this by using mass-curves.† He has also shown that, even without considering monthly temperatures, mass-curves could be constructed, using the predicted monthly run-off as ordinate increments, which would give proper values for the storage required.

The author over-emphasizes the importance of the long-term mean (pages 1138 and 1139). It is not the long-term mean which is so much to be desired, but the long-term record. To the writer, the term, "mean utilizable flow", is misleading. The most important thing to determine is the recurrence and interval of recurrence of series of dry years. For studying such problems, the mass-curve is by far the best and most convenient method.

As generally understood by engineers, evaporation is the difference between rainfall and run-off, and, as such, is capable of definite measurement on any water-shed where rainfall and stream-flow gauges are established. The author subdivides this evaporation into "evaporation from land surfaces" (to be corrected when there is a material quantity of water surface on the water-shed) and "transpiration". The writer believes this subdivision of the "losses out of rainfall" to be both an unfortunate and unnecessary complication of the subject.

If transpiration, which the author defines as "being the vaporization of water from the breathing pores or stomata of leaf and other vegetable

* Harrisburg, Pa.

† *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 346.

surfaces", were a quantity capable of measurement and determination on different water-sheds, much might be said in favor of differentiating between it and evaporation. Mr. Justin.

The transpiration of any particular water-shed is, at best, only a guess. No one has ever measured the transpiration of a water-shed. Even if the transpiration of all the different kinds of plants was determined with a fair degree of accuracy, it would be difficult to apply this information to the entire water-shed. The segregation of transpiration from evaporation from ground surfaces would not be simple. Thus, on page 1093, the author mentions the data on water requirements of crops, recorded in *Bulletins* of the U. S. Department of Agriculture, and states that "the great difficulty encountered in the above mentioned experiments was that of differentiating between the evaporation from the soil and transpiration".

The true condition of this subject of transpiration is well shown by the diversity of the opinions of authorities referred to by the author. On page 1095, he states "Some writers say deciduous trees use more water than grasses and grains, and others claim just the opposite." * * * "If plants transpired a quantity of water equal to from one-half to two times the evaporation from an equivalent surface of water, as claimed by some experimenters, a great many streams in the United States that have a very appreciable sustained flow would become intermittent."

The writer is skeptical of the assertion that "yields of hay, grain, etc., become a convenient index to the approximate relative quantities of transpiration to be expected on different water-sheds". As one farmer will obtain a greater yield of a certain crop under exactly similar conditions of soil and moisture, it would be necessary to take into account the character of the population. Perhaps, it might be possible to derive a formula for the transpiration on any given water-shed, in which the following factors, represented mathematically, would appear: Thriftiness of inhabitants (as a coefficient to be determined by judgment); tons of hay, bushels of corn, oats, rye, wheat, etc., density of population per square mile, percentage of farmers who are graduates of agricultural colleges, tons of fertilizer per acre, gallons of alcohol consumed per capita per year, etc., etc.

Considering all the facts so well pointed out by the author, it would seem to be almost impossible to measure the transpiration of any particular water-shed. He, however, attempts this very thing with his base curve of transpiration (Fig. 17). This curve closely approaches a straight line and makes the transpiration very nearly proportional to the increase in temperature above 40° Fahr. Although the author does not particularly emphasize the fact, it is clear that this curve is not based on observed data, but on the assumption, that "the law first

Mr. Justin. stated by Van't Hoff and Arrhenius, that most chemical reactions and physiological processes double in activity for every rise in temperature of 10° cent." is also applicable to transpiration.

Having thus obtained the theoretical transpiration for the different months, the author then modifies these figures by the use of a coefficient based on the "character and density of vegetation and length of growing season, with reference to temperature and hours of sunshine".

In a somewhat similar manner, curves are presented for the evaporation from land surfaces for various temperatures, and the results thus obtained for particular water-sheds are modified by the use of coefficients chosen by judgment based on the character of the water-shed.

In Tables 7, 9, and 11, the author has columns headed "Actual evaporation, in inches, loss from land area". Any one examining these tables would surely think that the figures represented the measured evaporation from the land surface of the water-shed, in inches. It is, in reality, nothing of the kind, but is merely the author's estimate of the number of inches, out of the actual difference between rainfall and run-off, which he believes should be credited to evaporation from the land surfaces.

Thus, it is demonstrated that the differentiation between evaporation from ground surfaces and transpiration is a matter which is still in the realm of surmise. Inasmuch as even the author believes that transpiration "must necessarily depend largely on the same factors as evaporation", and in view of the impracticability of differentiating, except within very wide limits, between evaporation from ground surfaces and transpiration, what practical purpose is to be served by the complication of this already complicated subject by the injection of transpiration?

In Table 5 is given a comparison between the mean annual run-off as computed by the author's method and as actually observed. The figures presented, except in the case of the McCloud River, show a startlingly close agreement between the observed and computed values, the discrepancy ranging as it does from a few hundredths of an inch to 1 in. This close agreement becomes the more remarkable when we consider the variation in the accuracy of the data involved. On some water-sheds, as pointed out by the writer,* the number of rainfall stations does not exceed 1 per 1 000 sq. miles of water-shed. Observations have generally been recorded by the native on whose property the gauge is placed. Sometimes these observations are made accurately and honestly; at other times they are not. Cases have come to the writer's attention where the observer has deliberately "faked" a whole month or more of rainfall records. Frequently, the observer removes the gauge to what he considers a more convenient location near his house and outbuildings, where the observations can be readily taken by

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 346.

his 12-year-old daughter. Generally speaking, enough money is not available to insure the efficient supervision of work of this class. The writer believes that engineers should not expect most rainfall records to be within 1 or 2 in. of the truth. In some cases the discrepancy is much greater. On some water-sheds where observations on rainfall have been started in anticipation of the installation of an important water-power or water-supply project, the supervision has been close, and the records are all that could be desired. In the past, even some of the records of run-off have not been any too accurate. The author's division of the difference between the recorded rainfall and run-off into evaporation from water surfaces, evaporation from land surfaces, and transpiration, is, as has been pointed out, pure surmise, within very wide limits. Mr. Justin.

When the author compiled Table 5, he had before him the recorded rainfall and run-off for the various water-sheds considered. He then manipulated his curves, his coefficient, and his judgment to derive the three quantities, evaporation from water surfaces, evaporation from ground surfaces, and transpiration, always bearing in mind the fact, subconsciously or otherwise, that the sum of the three must approximately equal the difference between run-off and rainfall. Nothing could be more simple.

The use of well-seasoned judgment in the prediction of run-off is most essential, but what assurance can the author give of making a reasonable prediction as to the transpiration on a water-shed where the run-off is unknown, when his figures for transpiration on water-sheds where run-off and rainfall are known are based on surmise, within very wide limits?

The author believes that there is no direct relationship between rainfall and run-off. The writer believes that, in his paper on the "Derivation of Run-Off from Rainfall Data", he proved that, within the limiting accuracy of the data involved, there was just such a relationship between rainfall and run-off, in the Northeastern United States, at any rate. He expressed this relationship by the formula, $C = 0.934 S^{0.155} \frac{R^2}{T}$, in which,

C = the annual run-off, in inches, on the water-shed.

R = annual rainfall, in inches, on the water-shed.

S = Slope of water-shed = elevation of highest point, minus elevation of lowest point, divided by the square root of the area.

T = Mean annual temperature.

Inasmuch as slope determines topography, and topography largely determines the character of the vegetal covering, this formula takes into account all the important factors which the author found to cause

Mr. Justin. variation in transpiration and ground-surface evaporation. This formula has the additional advantage of being based solely on observed data, and therefore, the writer believes, can be safely used throughout the territory to which it was applied.

If extended to the Middle West and Far West, it should, of course, be checked by additional observed data, and the constant and exponents modified, if necessary, in accordance therewith.

It must not be expected that on any given water-shed a certain annual rainfall will always have the same annual run-off. The discrepancy, as previously pointed out by the writer, can generally be accounted for by beginning the water year on a date at which the ground-water level is not the same as at the beginning of the other water years. If the annual temperature of the different years had been taken into account, as suggested by the author, the agreement would probably be still closer.

The writer is very strongly of the opinion that on any given water-shed, if the rainfall for two water years is the same, the run-off for the two years will also be practically the same, provided that ground-water and temperature conditions are similar.

Mr. Strong. A. M. STRONG,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer has read with much interest the full analysis of the factors governing run-off given in this paper. In too many cases in the past, efforts to compute run-off from precipitation have been unsatisfactory by reason of taking effect for cause. In using any formula or method for determining the relation between the two, a thorough knowledge of the laws governing evaporation, transpiration, and underground flow must be combined with a knowledge of the physical data for the drainage area under consideration. This information, accompanied by good judgment in comparing the known measurements of run-off with the records of precipitation, will give results of considerable value, where the proper coefficients are used, as shown by the author's computations. In any method, judgment and experience will largely determine the accuracy of the results, and are of more importance than the method used. In any case, the value of the results from computed run-off from any given drainage area can only be judged by comparison with known records. Great care should be used in comparing different drainage areas, even if adjoining, and no matter how similar they may appear.

Run-off from a given drainage area may be defined as that portion of the precipitation which remains on or near the surface of the ground too short a time to be lost or taken up by evaporation. A varying proportion of this is underground flow, and the remainder is surface flow; but that which is underground flow in one portion of

* Los Angeles, Cal.

the drainage area may become surface flow in another part. As the slow movement of underground water tends to equalize the seasonal variations in precipitation, the underground flow is fairly constant for any given drainage area, and largely governs the low-water flow of any stream. Flood flow is caused by surface run-off, and is governed by the irregular form in which the precipitation falls on the drainage area. Mr. Strong.

Evaporation and transpiration are very closely inter-related. Except in extreme cases, the character and extent of the vegetation on any drainage area are determined by the average quantity of water available for its growth. To a considerable extent, the variation in precipitation from year to year is met by variations in the extent and luxuriance of the vegetation, affecting both transpiration and evaporation. The ideal condition would be a continuous and even distribution of the precipitation during the growing season. Under these conditions, the character and extent of the vegetation would be such that it would take up all the water not lost by evaporation or carried off by deep underground flow.

Such an ideal condition is illustrated in the cienagas of the Southwest. These cienagas (springs or seepages) support a luxuriant vegetation consisting of a large variety of plant life. Usually, near the center, there is a little standing water, with its attendant plant growth, and, surrounding this, there are plants varying in kind and luxuriance with the quantity of water available for them. Where these cienagas occur on a side-hill, as is often the case, the extent of the vegetation is such that transpiration and evaporation take up all the flow. It is often possible to drain these cienagas by tunnels or wells, and, in cases which have come under the writer's observation, the quantity of water so obtained shows that the transpiration and evaporation over the entire area affected by the flow must have had an average of more than 100 in. per annum.

If flood-water run-off is considered to be caused by the irregular distribution of the precipitation, it is evident that the conditions governing the run-off from any single storm are so variable that any exact computation is impossible. Annual run-off, being to a large extent the total of the run-off from a number of storms, gives a more average condition. Any attempt to analyze this average condition must take into consideration all factors, climatic and physical, and then, at best, is only accurate in as far as a short-time average approaches the long-time average.

E. F. Chandler,* Assoc. M. Am. Soc. C. E. (by letter).—The author should receive much commendation for the careful and painstaking work which he has done in developing and presenting clearly this method of computing probable run-off. His method is based on Mr. Chandler.

* University, N. Dak.

Mr. Chandler.

a well-arranged and correct chain of reasoning, and therein differs from the methods customarily used in the past by many engineers—methods which the writer has seized every opportunity to criticize as misleading and incorrect.

The run-off is frequently regarded as a percentage of the rainfall. In the Eastern States, where careful consideration of the rainfall-run-off relation was first undertaken, and where the rainfall is so great that the run-off is usually more than half of it, such treatment did not lead to enormous percentage errors; but, in regions of smaller rainfall, it often leads to absurd results, and, in any region, it is wrong in principle. Although for convenience the run-off may sometimes be spoken of as a percentage, it is properly not a percentage, but a residual, the remainder of the rainfall after evaporation has occurred.

The author applies to different portions of the evaporation two different terms, evaporation and transpiration. Both rainfall and run-off can be measured with fair accuracy, and the evaporation from water surfaces can be measured with reasonably close approximation; but there is no practicable method for determining the true evaporation and transpiration over the ordinary land surface, except the indirect means of subtracting the run-off from the rainfall. Hence, the lateness of development of a well-founded method for making estimates of evaporation and transpiration without the direct use of run-off records.

The writer endorses heartily nearly every step in the author's method and (with possibly a few minor exceptions) every statement in his paper, but is led to further comments in emphasis of some points and in warning against difficulties which must be guarded against in the application. These came to his attention during an attempt to apply this method for exemplification and verification in several drainage areas, with the physical characteristics of which he is thoroughly familiar and in which he has supervised the maintenance of continuous run-off records for 10 years or more. These streams were the Red River of the North at Grand Forks, N. Dak., having a length of record of 33 years, and a drainage area of 25 000 sq. miles; the Red Lake River, at Crookston, Minn., 13 years' record, 5 800 sq. miles; the Mouse River, at Minot, N. Dak., 12 years' record, 8 400 sq. miles; and some smaller streams. The writer has not had time to complete these tabulations fully, but the points to which attention was brought especially by these comparisons were as follows:

1.—That this method is quite laborious as compared with the conventional and incorrect method of merely applying a percentage coefficient to the total annual rainfall; but the results are so good as to be well worth the trouble.

2.—That much caution is needful in determining the rainfall figures on which to base the computations for a very large drainage area.

Probably the only safe plan in studying such an area is to break it up into several smaller ones and compute each separately, for the normal rainfall differs so greatly in different portions of a large area that no average figures can logically be used. For example, the 25 000 sq. miles of the Red River area has a normal annual rainfall of 25 in. at the eastern and 16 in. at the western margin; as a result, the total annual run-off is ordinarily between 6 and 3 in. at the eastern edge, and between 1.0 and 0.2 in. at the western edge. No system of averaging rainfall figures from the different sections of the drainage can be properly used for deducing for this entire region a single series of rainfall figures appropriate for this purpose; the differences between different sections of the area are comparatively too great. Mr. Chandler.

3.—That if large lakes are in the drainage, lake storage must be taken into account, and this requires definite records of variations in lake level. For example, the Red Lake River, with a drainage area of 5 800 sq. miles, has a mean flow of about 1 600 sec-ft.; one-third of its area is above the outlet of Red Lake, and the area of the lake is 440 sq. miles. It is reported that, in the course of a few years, variations of 2 ft. occur in the lake level; this is a storage equivalent to one-third of the total normal annual flow of the entire drainage area of the river for a period of more than 16 months. The lake is in a timber country, mostly unsettled, or occupied only by scattered Indian settlers, and unreported changes in the discharge of the lake are likely to be caused by the temporary blocking of its outlet by sand-bars or drifting logs and brush, which, after a few months, would be washed away or passed around by the stream; if no lake records were kept, there might be unpredictable changes in the visible run-off farther down the stream, caused immediately by the lake, the ultimate relation of which to precipitation and temperature would be undiscoverable.

4.—The writer has often suggested that (in default of any more systematic and accurate basis of estimate), for each region, some figure might be determined as the measure of the annual rainfall usually needed to make good the evaporation and transpiration losses, and that any excess of rainfall above this figure may be expected to appear in the run-off. For Minnesota and the Dakotas, this figure would be 20 in. per year, or thereabouts. This assumption is more nearly correct than to estimate the run-off as a fixed percentage of the rainfall, but it must not be assumed to be an accurate statement or anything better than a mere makeshift basis for rough estimates; topography, seasonal distribution of rainfall, rate and number of heavy rainfalls, and temperature, ought to be considered, as the author has done.

It is unquestionable, however, that in the extreme Northern States, if the annual precipitation is much less than 20 in., the run-off becomes almost inappreciable if the rains are well-distributed and not violent

Mr. Chandler.

and do not occur when the ground is frozen. In other words, after evaporation and transpiration losses, the remainder reaching the ground-water level is there so very small that scarcely any ground-water flows into the streams. For example, the Mouse River, at Minot, N. Dak., drains 8 400 sq. miles of country between latitudes 48° and 50°, the average rainfall for which is about 15 in. As remarked by Mr. Meyer, practically all the water in the usual stream in this latitude and region, between November 1st and March 1st, is seepage flow; in this case, the seepage flow, except after a season of unusually heavy rainfall, is so small that in some winters the river almost disappears, often having for months a flow much less than 5 sec-ft.; the long-period average (more than 10 years) for the 6 months from October to March, inclusive, is only 26 sec-ft., which is less than 0.03 in. of total run-off in the 6 months.

5.—That in the use, for any stream, of the method elaborated by the author, it is necessary to assume a coefficient by which the evaporations, as taken from the curve, are multiplied. This assumed coefficient is verified or revised by comparison of the figures secured by its use with those of actual measured run-off of the same stream, or a similar one in a near-by region. Unless such records of actual measurements of run-off are available, there is no check on the correctness of the assumed coefficients. In a region, such as the Northwestern Central States, where evaporation and transpiration take up nearly all the precipitation, an error as small as 5% in the assumed coefficient might double or triple the computed run-off, or might wipe it out entirely. Hence, it is evident that great care and caution must be exercised in using even this most logical and excellent method. In such a region of small run-off, unless actual records are available on the same or closely adjoining streams, so that the adopted coefficient can be verified by extensive comparison, no computations by its use ought to be permitted, for the results would be estimates so very rough that some method apparently much less refined, or even mere guesswork, would be likely to give as dependable results. Some actual run-off records are absolutely necessary as a starting point for the application of this new method, in extending estimates into periods during which only precipitation and temperature records are available; otherwise, the computer is almost helpless.

This last comment is not intended to convey any general objection to the new methods, but rather as an expression of the writer's feeling that the continuance of gauging station and run-off records, based directly on actual measurements of flow, will be found to be more necessary and also more profitable than formerly; their value will be found to be much increased if this new method by which such advantageous extensions of run-off records can be deduced, comes into general use.

C. E. GRUNSKY,* M. AM. SOC. C. E. (by letter).—Referring broadly to practically any portion of the United States, it can be stated that wherever the annual rainfall exceeds 40 in., the portion thereof which will be lost to the stream by evaporation, transpiration, etc., is fairly uniform. The soil takes and gives to the air and to the plants a quantity of water which increases up to a certain limit with increasing quantities of rain. When the annual rainfall has reached from 40 to 50 in., however, there will be but little probability that larger quantities will cause any material increase in the evaporation from a soil already subject to frequent saturation, or that more rain will greatly increase the transpiration losses. If the rain is greater in quantity, there will be more rainy days and greater mean annual humidity—causes tending to prevent any considerable increase in the quantity of water which evaporation will keep out of the stream. Mr. Grunsky.

In other words, there is an astonishing uniformity (average values alone being considered) in the quantity of water which will be lost to the stream whenever the rainfall approximates or exceeds 40 in. per annum. This fact was pointed out by the writer long ago, and has guided him in formulating the following rule for approximating from the seasonal (12 months) rainfall the probable run-off:†

"The percentage of the seasonal (12 months) rainfall which appears in the stream as run-off, when the rain is less than 50 in., is equal to the number of inches of rain. When the seasonal rain is in excess of 50 in., 25 in. thereof goes to the ground (evaporates, etc.), the remainder is the run-off."

According to this rule, the run-off for 10 in. of rain in 12 months would be estimated at 10%, or 1.0 in.; for 30 in. of rain, it would be 30%, or 9 in.; for 60 in. of rain, it would be 60 — 25 = 35 in.

This rule will give fairly close results for practically any portion of the United States, but, in using it or any run-off curve, it must be remembered that probabilities only are to be determined, and that the smaller the seasonal (12 months) rainfall the greater may be the percentage departure of the actual from the estimated run-off in any single period of 12 months.

Revised studies made by the writer of available run-off data in California, where the rains which produce the seasonal run-off fall between October 1st and May 1st, have led him to adopt the curve shown by Fig. 37, which departs somewhat from the curve based on the foregoing rule, as best expressing the relation (under the Pacific Slope climatic conditions) of probable run-off to the seasonal rain.

It will be noted that the total quantity credited by this curve to evaporation, transpiration, and other losses is 28 in. per year for quantities of rain in excess of 50 in. A curve for run-off under

* San Francisco, Cal.

† *Transactions*, Am. Soc. C. E., Vol. LXI, p. 514.

Mr. Grunsky. Eastern climatic conditions would probably show 25 in. of loss for rain exceeding 50 in. in 12 months. For a seasonal precipitation of less than 50 in., the curve here presented, or a slight modification thereof, will fulfill every ordinary estimate requirement where, from a known rainfall, predictions of the total run-off in 12 months are to be made.

The writer's experience indicates that it is usually difficult to determine the rain on the water-shed. Such curves as shown by him on Plate LIX, of Vol. LXI of *Transactions,* are by no means conjecture. They show the great effect of orographic features on rain distribution, and can be used to show the small dependence to be placed on the determination of rain from the recorded quantities at a few irregularly distributed stations.

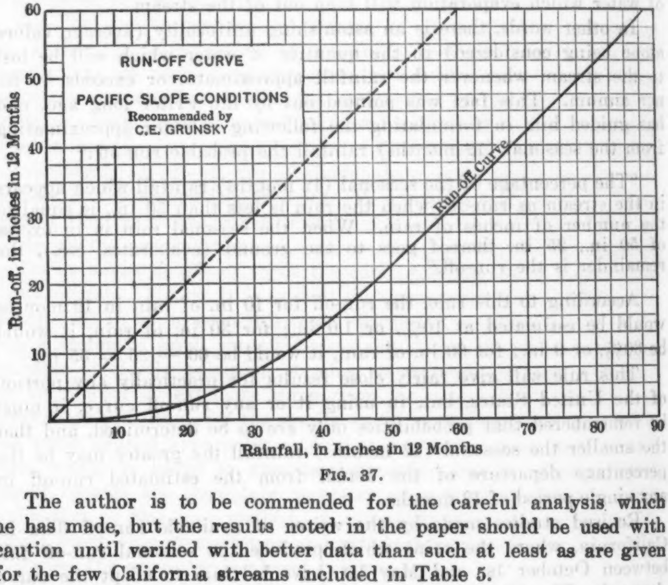


FIG. 37.

The author is to be commended for the careful analysis which he has made, but the results noted in the paper should be used with caution until verified with better data than such at least as are given for the few California streams included in Table 5.

Mr. Horton. ROBERT E. HORTON,* M. AM. SOC. C. E. (by letter).—Recognizing the fact that run-off is the result of many factors besides rainfall, the writer has for many years avoided its estimation from rainfall alone where possible. As far as he has gone, he has reached the following conclusions:

1.—Under the present state of the art, fairly accurate estimates of total annual run-off can be made in most cases from data that

* Albany, N. Y.

are generally available, namely, rainfall, temperature, and the physical characteristics and cultural conditions for the drainage basin. Mr. Horton.

2.—From the data usually available, it is much more difficult to make accurate direct calculation of the monthly or daily run-off for individual months than of the yearly average.

3.—From existing gaugings of streams, the percentage distribution of the run-off throughout different months, or other subdivisions of the year, can be determined approximately for different classes of streams and for different localities.

4.—Having given the total annual run-off and the law of monthly distribution, a good and fairly correct estimate of run-off, month by month, can be obtained.

This method of determining the relative monthly distribution of run-off has been designated the "distributive method", by the writer, to distinguish it from others by which an effort is made to ascertain the regimen of a stream, without actual gaugings.

It is a fact that the total run-off from a drainage area can be estimated with greater ease and accuracy than the run-off for the individual months. This is due to the fact that many elements, not taken directly into account in such calculations, are mutually counterbalanced in a large measure in the course of the annual cycle. This fact, that the annual run-off can be calculated with greater accuracy than the individual monthly run-off, led the writer, some years ago, to use for the latter purpose what may be described as a "distributive method". Mr. Meyer follows a somewhat similar procedure.

The writer has for years firmly believed that much better estimates of run-off could be made than have generally prevailed heretofore, and that such estimates should depend, not only on rainfall, but on other physical data pertaining to the drainage basin. Such procedure may be described as a "hydro-physical method" as distinguished from methods dependent on rainfall as the only or chief factor. Mr. Meyer's method involves ingenious ways of taking into account approximately the more important factors affecting run-off. Like all such methods, it is subject to revision, refinement, and elaboration, but the writer believes that some more or less similar hydro-physical method will shortly come into use among engineers for the solution of the extremely important problem of estimating run-off in the absence of actual gaugings. The author's discussion of the exceedingly complex phenomena involved in the hydrologic cycle is unusually lucid and accurate, although necessarily quite incomplete, and some data are quoted which are subject to misinterpretation.

Experiments by Transeau are cited which appear to indicate greater evaporation from a bare gravel slide than from any other condition of the soil. In these experiments there was a saturated medium

Mr. Horton.

exposed freely to evaporation all the time. Actually, the evaporation opportunity for the gravel slide is of short duration, and in some cases is continuous for the other soil conditions cited. The rate may be greater, but the total evaporation is less for the gravel slide than for the other conditions.

Methods of estimating run-off in the absence of actual gaugings are not uncommon, and, the writer believes, are a disgrace to the Profession. The only excuses which can be offered in some such cases are (1) ignorance, (2) laziness, (3) an intention to deceive. The first, the writer is thankful to say, is by far the most common. Hydrology is a new, vast, and rapidly growing science, and that the majority of engineers, busy mostly with other matters, should not be familiar with all the details of such a complicated subject is natural, and would be expected. We do not expect all engineers to be experts in subaqueous foundations, electric dynamo design, or isostacy. This criticism is not aimed at any one in particular, nor is it suggested by Mr. Meyer's paper or its discussion. It is given as the explanation of a condition.

Confining attention to methods of estimating year by year the annual run-off of a drainage basin in the absence of gaugings, we may classify as follows:

1.—The standard river method;
2.—The direct comparative method;
3.—The rainfall percentage method;
4.—The empirical method; and
5.—Hydro-physical or rainfall-loss methods.

The Standard River Method.—The standard river method consists essentially in adopting the results obtained for some stream which has been accurately gauged for a long period of years as applying generally without modification to any stream in the locality. This method is mentioned only to condemn it. In early days, when both stream gauging and run-off data were meager, and hydrologic principles not fully understood, this method gained wide acceptance, notably in the case of the use of the Croton and Sudbury River gaugings as a basis for estimating the yield of other streams under all sorts of conditions.* With present knowledge of the effect of different physiographic and cultural conditions on run-off, the adoption of the yield of any stream as applying to any other stream without a study of the comparative conditions of run-off for the two areas is unjustifiable, even in the absence of all data except those which can be obtained from an inspection of the water-shed.

* As an illustration of recent advocation of this method, see article by W. L. Church, "Formulas and Computations for Horse-Power Value of Streams." *Engineering Record*, July 1st, 1905, pp. 11-12.

Mr. Horton.

In the absence of a good stream, well gauged, and having similar meteorologic and hydrologic conditions for the application of the comparative run-off method, and even as a good check on that method, the use of the rainfall-loss method is desirable.

The Direct Comparative Method.—In the direct comparative method, the measured run-off per square mile from a drainage area that has been gauged is assumed to apply directly, *pro rata,* to the area in question. This method is simple of application, and in many cases is no doubt as good as any, if not the best. Cases where its use is certainly legitimate arise where a stream has been gauged at one location, and its yield at another near-by location is to be determined. One of the two areas always includes the other. All the physical conditions are the same for both over a considerable proportion of the larger area, even though the physical conditions, rainfall, culture, etc., may be quite different in the part of the area not common to both. Even where the area of which the yield is to be determined comprises the whole or a part of the area that has been gauged, the direct comparative method may require correction to avoid serious errors. For example: A stream has been gauged at a point where it emerges from the mountains, its drainage area being 100 sq. miles. Its yield is desired at a point on the plains below, where its drainage area is 150 sq. miles. The measured run-off for the upper area is, say, 30 in. By the direct comparative method, this would be assumed as the run-off rate for the entire area; but, suppose that, owing to differences in rainfall, slope, and cultural conditions, the actual run-off rate for the 50 intermediate square miles was only 20 in., then the true run-off for the 150 sq. miles would be $\frac{2}{3} \times 30 + \frac{1}{3} \times 20 = 26.67$ in.

The Rainfall Percentage Method.—In using the rainfall percentage method, the percentage of run-off to rainfall is determined from actual gaugings for some area or areas for which the rainfall and all the physical conditions are assumed to be the same as for the area for which the run-off is to be estimated. This method has a weakness in common with the direct comparative method, in that, when it has been applied heretofore, it has been the exception rather than the rule that the engineer applying it really knew that the physical conditions affecting run-off (other than rainfall) were the same or even essentially the same for the two areas. If he had known these facts in sufficient detail, he should have applied a correction (if any was needed) to the run-off of the intermediate area in using the direct comparative method, and he would not have applied the rainfall percentage method at all, unless for a crude, rough approximation.

The rainfall percentage method has another inherent weakness, in that run-off is a residue from rainfall, not a ratio to rainfall. It is what is left after certain natural losses have been deducted. Some

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of these losses, such as interception, are more or less proportional to the rainfall, and so far may be treated as ratios; other losses are either quite independent of rainfall, as evaporation and transpiration, or are dependent to an equal or greater degree on other factors. Soil evaporation comes in this class.

Run-off Formulas.—As to run-off formulas: Such of these as involve rainfall as the only factor belong in a class with the rainfall percentage method. They are nothing more than supposed aids to judgment in selecting the proper percentage (Grunsky's). Others take into account certain other factors, as slope (Justin's), and temperature (Vermeule's and Justin's).

Probably no simple formula can be derived which takes into account in a satisfactory manner all the factors of prime importance which affect annual run-off. Methods undoubtedly can be evolved which take all these factors into account.

The word "method" is here used in distinction from the word "formula", as indicating greater flexibility. In a "formula", the factors are fixed, though their values may vary. In a "method" of estimating run-off, the factors used may be different in different cases, according to the data available, and yet the method retains its identity. Hydrology is largely a matter of methods. The guiding principle is that use should be made of all the available data, if the value of the result justifies the labor involved.

Rainfall-Loss Methods.—The method of estimating run-off by deducting the estimated rainfall losses from the measured rainfall is not new; it is the method customarily used in England, and is there often preferred to actual gaugings, the writer thinks without reason. The usual English application seems to the writer very crude; yet, considering the simple soil, cultural, and climatic conditions of the chalk and certain other regions from which water supplies are commonly derived in England, it becomes evident that the necessity for more detailed estimates is much less imperative than in the United States.

Larger drainage basins, less numerous rainfall records, and more complex soil and cultural conditions, are causes which serve to explain the less frequent application of this method in the United States as compared with England.

The late George W. Rafter, M. Am. Soc. C. E.—from whom, by the way, Mr. Justin got the suggestion for his own run-off formula—made some quite extensive applications of the rainfall-loss method, taking into account area and extent of different cultural conditions in the drainage basin and relative water losses for each. In 1904 the writer used a rainfall-loss method to explain the difference in the observed run-off of streams in the agricultural, forested, and barren deforested regions of Michigan. Mr. Rafter's and the writer's methods

differ from one another, from the English, and from the author's, but the underlying principle of all is the same. Mr. Horton.

As developed in 1904, the writer subdivided rainfall losses into three parts:

1.—Interception;
2.—Transpiration; and
3.—Soil evaporation.

Mr. Justin criticizes Mr. Meyer's method on the ground that it does not take into account the slope of the drainage basin. This is not necessarily true—slope is one of the factors affecting run-off, but its effect is probably less than usually considered to be the case. The rainfall being correctly determined, slope may be considered as one of the influences affecting the water losses. Slope of the surface does not materially affect the interception loss. It has but small effect on the transpiration loss in humid climates where the normal ground-water supply is adequate for plant growth, except perhaps in the case of very steep slopes; because experience shows that crops grow as well on moderately rolling ground as on plateaus at the same average elevation. In general, where infiltration exceeds plant requirements, the transpiration loss is little affected by slope of the surface. Slope does, however, materially affect the soil surface evaporation. For the same exposure to air and sun, the evaporation rate will not be affected by slope, but as the water does not stand on the ground as long on steep slopes as on flat lands, the duration of surface evaporation—or, as the writer prefers to call it, the "evaporation opportunity"—is reduced by increased slope. Soil evaporation, being the product of rate and duration, the total amount of this loss is directly affected by slope. Thus slope affects only one of the three principal sources of rainfall losses. Mr. Meyer's method includes an evaporation coefficient dependent on the physical characteristics of the drainage basin, and the effect of slope may very properly be included in selecting this coefficient.

Effective Slope.—Soil evaporation, as just explained, is affected by all the elements affecting water surface evaporation. In addition, it is influenced by:

1.—The quantity and distribution of rainfall;
  a.—Reduction in rainfall reaching the ground through interception;
  b.—Shading of the soil surface from sun and wind;
  c.—Change in the absorptive and capillary character of the soil by tillage;
2.—Extent and density of vegetation;

Mr. Horton.

3.—Slope of the ground surface;

4.—Depth and porosity of the soil; and

5.—Orientation.

It appears that soil evaporation is so complex that it cannot be reliably estimated in terms of temperature as the only variable.

Some experimental indication of the soil evaporation loss is obtainable from lysimeter experiments. Data of crop area, normal growth curves, and tables of duration of the growing season, afford a basis of estimating the amount of interception and soil shading, and experiments in forest meteorology furnish light on this phase of the subject. For the present, it appears to be necessary to include a coefficient or arbitrary factor, derived by judgment from a consideration of all these factors, in estimating soil surface evaporation.

Certain advantages of the hydro-physical method of estimating stream flow appear to be as follows:

1.—Run-off is treated, not as a ratio, but as a difference between water supply and water losses, which it is in nature.

2.—It takes into account a much larger number of the factors affecting stream flow than do other methods.

3.—It is logical, in that it makes use of all the available information bearing on the subject.

4.—The selection of arbitrary factors or coefficients by judgment does not apply to the entire estimate, but is, or may be, limited to part of the factors, the other factors being based on direct observation or experiment.

Advocates of the rainfall-percentage method of estimating run-off, or of particular run-off formulas, will lose nothing by possessing themselves of the added knowledge of the drainage basin, needed in making an estimate of run-off by the hydro-physical method, and by so doing they may avoid committing gross blunders, such as have sometimes come to the writer's notice, where the direct comparison or rainfall-percentage methods have been used.

Transpiration is an exceedingly complex phenomenon. The writer does not think it is adequately covered or accounted for by Mr. Meyer's method.

The existing data of transpiration by crops are summarized in Briggs' and Shantz' papers, which, taken in conjunction with a census of crop production on the given area, seem to the writer to furnish the best possible basis for estimating this loss. Crop yield is not a precise measure of transpiration loss, but it seems to the writer that it is the best and simplest measure generally available. Transpiration and crop production both depend jointly on light, temperature, humidity, wind velocity, rainfall, and other factors. The yield of the crop is a measure of the resultant effect of all these factors on

the vegetative activity of the plant. Transpiration is closely allied to the vegetative activity of the crop, and so the yield becomes a measure of this also. In using transpiration as a factor in stream-flow estimation, a number of the most complex and troublesome variables affecting stream flow are automatically taken into account. Mr. Horton.

Mr. Justin questions the possibility of securing data of transpiration losses independent of soil evaporation. He will do well to consult the results of the extensive experiments by Briggs and Shantz, J. W. Leather, and others, in which the transpiration loss for all sorts of common crops has been very accurately determined, independent of soil evaporation. The results are expressed in terms of the transpiration ratio or weight of water transpired per unit weight of dry grain or dry matter produced.

Fig. 19 must be considered as a highly generalized and only approximate average curve of transpiration.

The author's method divides water losses into two principal classes: "transpiration" and "evaporation". The writer prefers to use three: interception, transpiration, and soil evaporation. Interception varies with rainfall and with rainfall distribution. Its variation with temperature and yield of crop, though probably appreciable, is relatively small, compared with the total, and may be considered as of the secondary order, and usually negligible for purposes of practical calculation. Interception is zero for zero rainfall, whereas transpiration and soil evaporation may be considerable for a month of no rain.

W. G. Hoyt,* Assoc. M. Am. Soc. C. E. (by letter).—After making a careful study of the method of determining run-off from rainfall and other physical data, and after computing the "precipitation minus loss" and comparing it with the run-off of several Wisconsin streams, the writer feels that Mr. Meyer's method is not only rational, but is the best one devised to date. This paper gives convincing evidence of the great length of time spent in the study and computations. The writer welcomes a method which does not involve the determination of a factor for use in a formula of which precipitation is one factor and run-off is the result, but which attempts, instead, to ascertain the actual losses from precipitation, in order to determine the residual or run-off. Mr. Hoyt.

Of the prior methods devised to compute run-off, the writer believes that that of Vermeule† is the most rational, because, in it he attempted to do roughly what Mr. Meyer has apparently done in detail. The writer hopes that other engineers will be sufficiently interested to measure and compute the flow of rivers in different parts of the country and compare the results, in order, not only to decide the

* Madison, Wis.

† "Water Supply of New Jersey," 1894; and Annual Report, State Geologist of New Jersey, 1899.

Mr. Hoyt. accuracy of the method, but to ascertain what corrections and additions are needed. The writer feels that he cannot justly criticise Mr. Meyer's paper without comparing computed flow and measured flow at a number of stations.

In his introductory paragraph Mr. Meyer states that "there is need for a method of computing run-off from other physical data for the purpose of extending and supplementing short-term stream-flow records." It should be noted that he does not suggest that his method be used to compute the run-off of streams for which there are no records of observed run-off. To the layman the tables of run-off computed by Mr. Meyer's method would seem to be all that could be desired, whereas, without coefficients carefully derived from observational data, those tables might be far from accurate. Many expensive engineering works, or their remains, found in various parts of the country represent misplaced confidence in engineers or others who have thought it possible to predict stream flow with few or no base data.

At the beginning of this discussion the writer wishes to quote from the introduction to recent papers on the surface water supply of the United States published by the United States Geological Survey:*

"Even though the monthly means for any station may represent with a high degree of accuracy the quantity of water flowing past the gage, the figures showing discharge per square mile and depth of run-off in inches may be subject to gross errors, which result from including in the measured drainage area large noncontributing districts or omitting estimates of water diverted for irrigation or other use. 'Second-feet per square mile' and 'run-off (depth in inches)' have therefore not been computed for streams draining areas in which the annual rainfall is less than 20 inches, nor for streams in which the precipitation exceeds 20 inches if such computations might probably be uncertain and misleading because of the presence of large noncontributing districts in the measured drainage area, of omitting estimates of water diverted for irrigation or other use, or of artificial control or unusual natural control of the flow of the river above the gaging station. All values of 'second-feet per square mile' and 'run-off (depth in inches)' previously published by the United States Geological Survey should be used with extreme caution and such values in this report should be used with care because of possible inherent sources of error not known."

On account of such possible errors in the determination of "run-off (depth in inches)" from the drainage basin, it should be noted that these figures have been omitted where the Survey was reasonably sure that the entire basin was not at all times a contributor to the run-off. In certain sections of Minnesota, especially the western and northwestern parts, and also in northern Wisconsin, there are large swamp areas which contribute to the run-off of the streams only in

* U. S. Geol. Survey, Water-Supply Paper 353, p. 15.

times of high water. Such swamps and other non-contributing areas have considerable bearing on the determination of the coefficients, and any engineer attempting to use Mr. Meyer's method should study the drainage area very carefully. The writer, due to the fact that he was not well enough acquainted with the details of the method, did not base his coefficients so much on a previous study of the area as on the results he wished to obtain. Mr. Hoyt.

Mr. Meyer's method determines the "precipitation minus total losses, in inches," which, over a long period of years, approximately represents the run-off, if his coefficients based on the observed run-off are correct. As many of the published figures of run-off per square mile are subject to gross errors, they should be used with extreme care, and in determining a coefficient there should be a recomputation from the mean monthly flow and the actual contributing area, rather than the total area of the drainage basin above the gauging station.

It should be noted, also, that the records of stream flow presented in recent water-supply papers cover a "water year," beginning October 1st and ending September 30th, instead of the calendar year. The fact that the water year included by these dates is not strictly applicable throughout the entire country, and may even vary in the same locality from year to year, is of course recognized. No doubt exists as to the immense quantity of seepage flow in the central northwestern part of the United States, and for this reason Mr. Meyer's method of comparing the run-off during the period from March 1st of any year to February 28th or 29th of the following year, with the precipitation that occurs from November 1st of the preceding year to October 31st of the given year, is logical; it will be noted, however, that for some of the Wisconsin streams the computed run-off from November 1st of the preceding year to October 31st of the given year agrees more closely with the observed run-off for a similar period than the results shown in the summary of data and computations for the various drainage basins, most of which represent the run-off for the year beginning March 1st of a given year and ending February 28th or 29th of the following year. This is especially true of the Upper Wisconsin River, and is no doubt due to the large surface run-off during the period, November to February, inclusive.

As to the errors in the data on precipitation, much could be said. Undoubtedly many of the United States Weather Bureau Stations are seldom visited, and receive little supervision from the local offices. Considering the small area of the rain gauge and the large area over which the records apply, the estimates of monthly rainfall at a number of different stations in a drainage area agree surprisingly well. If the error introduced by the use of such records is at all constant, it is provided for in deriving the coefficients. Though the location of many Weather Bureau stations with respect to buildings and

Mr. Hoyt. trees is unfavorable for the determination of the precipitation in the surrounding country, it is doubtful if they are often moved, so that the error from year to year should be nearly the same.

In connection with his study of evaporation, Mr. Meyer has prepared a curve, Fig. 8, on which he has plotted the mean observed evaporation at University, N. Dak., the mean observed evaporation at Grand River Lock, Wis., and the mean computed evaporation at St. Paul, Minn. It is noted that the results of studies of evaporation records at Madison and Menasha, Wis., and at Iowa City, Iowa,* did not plot on this curve. As Mr. Meyer had access to these records, the writer would like to know why he did not use them in his determination of the evaporation, or at least plot the values on his curve. His evaporation curve from land and water is approximately the same as the formula used by FitzGerald and others, and the writer is somewhat surprised to find that this formula gives results so comparable with observed results, especially those taken at Mount Hope, N. Y., and Boston, Mass.

In the ordinary drainage basin, however, there is so little water surface that it is seldom necessary to take into account the loss from evaporation from such surfaces, but apparently this curve enters into Mr. Meyer's main curve for determining evaporation from land areas for various temperatures and rates of rainfall. The writer would like more detailed information in regard to the development of this curve; he would also like information as to the basis for the many assumptions that apparently had to be made.

The writer is of the opinion that run-off from rain falling in short intervals will differ radically from the run-off from the same quantity of rain well distributed over the month. Naturally, Mr. Meyer could not develop a curve that would take into consideration all these conditions, and it is supposed that his curve represents ordinary conditions of well-distributed precipitation.

No experiments have given conclusive information concerning actual losses by transpiration, and apparently Mr. Meyer's base curve is founded to a large extent on Van't Hoff's law. A loss determined by this curve, based on Van't Hoff's law and multiplied by a coefficient necessary to reduce it to what is called the normal seasonal transpiration, is as accurate as any. For a time the writer could not see why the losses from transpiration and evaporation could not be grouped into one loss to which one coefficient could be applied, but, as such losses may be accurately determined by future experiments, the method of using two coefficients is undoubtedly better.

The writer regrets that Mr. Meyer has not given, for each drainage basin for which he has determined the run-off, the coefficient he used

* "Water Resources of Minnesota," 1909–1912, State Drainage Commission, pp. 555–564.

to reduce the values of transpiration obtained from the transpiration curve to the so-called transpiration loss, and the evaporation from land surface obtained from the evaporation curve to the so-called evaporation loss from land surface. (The writer has used the word "so-called" in this connection because the quantities as determined from the evaporation and transpiration curves may not of necessity be strictly evaporation and transpiration losses.) It is hoped that Mr. Meyer will do this in his closing discussion, in order that those interested may see at a glance the relation between these coefficients and the extent to which they vary from year to year. Undoubtedly the accuracy of the method depends, not only on the accuracy of the coefficient, but also on the applicability of the same coefficient year after year, or so long as there are no radical changes in the drainage basin, such as marked deforestation, or the construction of large storage reservoirs. Apparently there is no reason why the coefficient should vary to any extent, but the writer is of the opinion that more than 2 or 3 years of records are necessary for its determination, as in order to use the data of 1 or 2 years' run-off it is necessary to make a careful study to ascertain whether during the year there was considerable seepage flow from the preceding year, or whether a large quantity of water was absorbed to make up for low ground-water during preceding years. Mr. Hoyt.

In order to determine approximately the variation of coefficients in different drainage areas, and the accuracy of the method, the writer has applied Mr. Meyer's method to several drainage basins in Wisconsin. These data and computations, together with a brief description of the characteristics of each of the basins, form part of this discussion. (See Tables 38 to 48.)

The coefficients used in the tables to reduce the values taken from the curve to those which would give the most consistent results are given in Table 36.

TABLE 36.

Drainage basin.	Coefficient.	
	Transpiration.	Evaporation.
Rock River Basin above Rockton, Ill.	0.75	1.05
Black River Basin above Neillsville, Wis.	0.70	0.88
Wisconsin River Basin above Rhinelander, Wis.	0.62	0.59
Wisconsin River Basin above Merrill, Wis.	0.66	0.66
Wisconsin River Basin between Merrill and Rhinelander, Wis.	0.66	0.58

Provided the values as taken from the evaporation and transpiration curves for the year were added together, it would have been necessary to use the coefficients given in Table 37 to reduce the quantity (obtained

Mr. Hoyt. by subtracting the sum of the evaporation and transpiration losses from the precipitation) to a figure which would be consistent with the observed run-off.

TABLE 37.

	Mean.
Rock River Basin above Rockton, Ill.	0.98, varies from 0.85 to 1.31
Rock River Basin above Neillsville, Wis.	0.88, " " 0.73 to 1.02
Wisconsin River Basin above Rhinelander, Wis.	0.64, " " 0.42 to 0.76
Wisconsin River Basin above Merrill, Wis.	0.63, " " 0.49 to 0.72
Wisconsin River Basin between Merrill and Rhinelander, Wis.	0.64, " " 0.58 to 0.72

Mr. Meyer states, on page 1100:

"To the values of evaporation, in inches of depth per month, as taken off the curve, a coefficient must be applied to reduce these quantities to actual evaporation from the given water-shed. This coefficient ranges from about 0.95 to 1.25 for most water-sheds of the Northwest, and for similar ones elsewhere. The coefficient to be used depends on topography, vegetal cover, soil, subsoil, humidity, and wind. An extremely high coefficient of evaporation would result from flat topography devoid of vegetation, moderately pervious, shallow soil underlain with impervious subsoil or rock, low humidity, and high wind velocity. An extremely low coefficient would result from rugged topography, bare scanty soil underlain with rock, high humidity, and low wind velocity. Between these extremes the usual working values will be found. With a little experience, one can select coefficients for different water-sheds with considerable accuracy."

It should be noted that the writer finds an apparent range in coefficients from 0.58 to 1.05, instead of from 0.95 to 1.25. The large range in coefficient for such a small territory as Wisconsin is very unfortunate, but perhaps Mr. Meyer, by reason of his long use of the method, may be able to discover the reason for the large variation in coefficients obtained by the writer, or can find causative errors in the computation. The large variation in coefficients for the different basins shows the necessity of basing the coefficient on actual determinations of flow, and the range of values of the coefficients for the same basin emphasizes the need for records covering several years in order to obtain a mean coefficient that will eliminate the effects of seepage and storage. The run-off from the area between Rhinelander and Merrill was computed to ascertain if there was any radical change in coefficient due to the exclusion of the large part of the drainage area above Rhinelander that is controlled by reservoirs. Apparently, the operation of these reservoirs did not materially affect the total run-off, although its distribution has undoubtedly been changed. The writer has not attempted to compute monthly run-off for any of the streams, as he

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feels that any computation of the monthly run-off by using the curves must be largely a matter of judgment in adding to and subtracting from the "precipitation minus losses" as determined from the various curves. It should be noted that Mr. Meyer gave no information concerning the construction of his curves for the flow of the Root River at Houston, Minn., to aid in the determination of surface run-off resulting from a monthly curve of "precipitation minus losses." Mr. Meyer would add to his already very valuable paper by giving detailed information relative to the development of these curves. In order to show the relation between the run-off computed from "precipitation

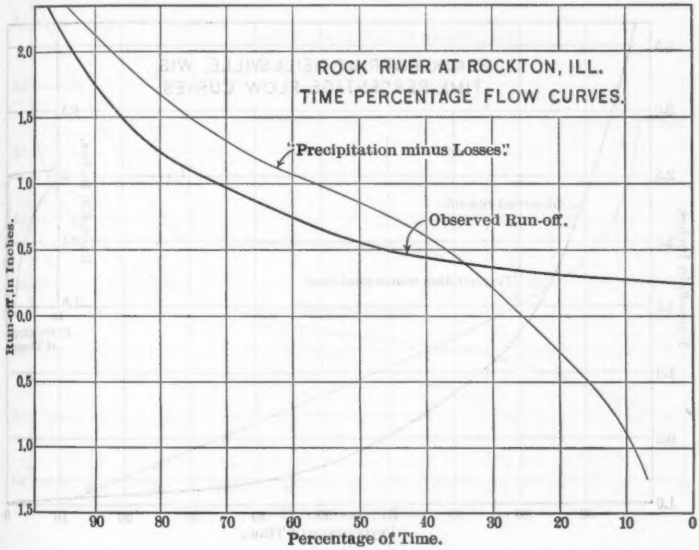

FIG. 38.

minus losses" and the actual run-off, curves showing the percentage of time covered by actual and computed records of run-off in inches for Rock River at Rockton, Ill., Fig. 38, Black River at Neillsville, Wis., Fig. 39, and Wisconsin River above Rhinelander, Wis., Fig. 40, are presented herewith.

These curves are the same as Mr. Meyer's frequency curves of run-off, Fig. 33. Apparently his frequency curves represent observed run-off and not "precipitation minus losses". It will be noticed that where there is very little ground storage, as on Black River at Neillsville, Wis., the percentage of time in which the actual run-off

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is less than that computed from "precipitation minus total losses" is greater than where there is large storage, either natural, as on Rock River at Rockton, Ill., or natural and artificial, as on Wisconsin River at Rhinelander, Wis. For the station on Black River at Neillsville, Wis., the observed run-off is less than that computed from "precipitation minus losses" for about 70% of the time; on Rock River at Rockton, Ill., the observed run-off is less than the computed run-off for 66% of the time; and on Wisconsin River at Rhinelander, Wis., where there is large natural and artificial storage, the observed run-off is less than the computed for only 34% of the time.

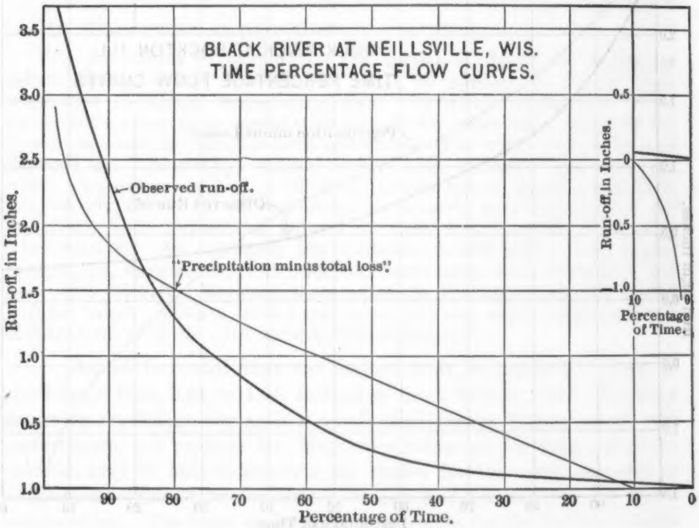

FIG. 39.

Until a very detailed study of a drainage basin has been made, it is doubtful whether the run-off computed from "precipitation minus losses" for any particular month can be translated into the observed run-off for that particular month with any great degree of accuracy. The writer realizes that Mr. Meyer's determination of the monthly flow of Root River in Minnesota checks remarkably close with the observed flow, so close in fact that he has been able to show the writer, under whose direction this gauging station has been maintained, errors in the observed flow. Knowing the characteristics of the flow of Root River as the writer does, he realizes that the determination of the

monthly flow of any river may be possible. He desires, however, more proof before his doubt changes to belief. Mr. Hoyt.

For this reason the writer prepared the curves on Figs. 38 to 40 in order to ascertain what results could be obtained mathematically without making any adjustments for ground storage or seepage flow. The tables show in general, that the observed run-off in any one month is larger than that computed from "precipitation minus losses," at least for Wisconsin streams, from about March 1st to August 31st, and that the run-off is less than "precipitation minus

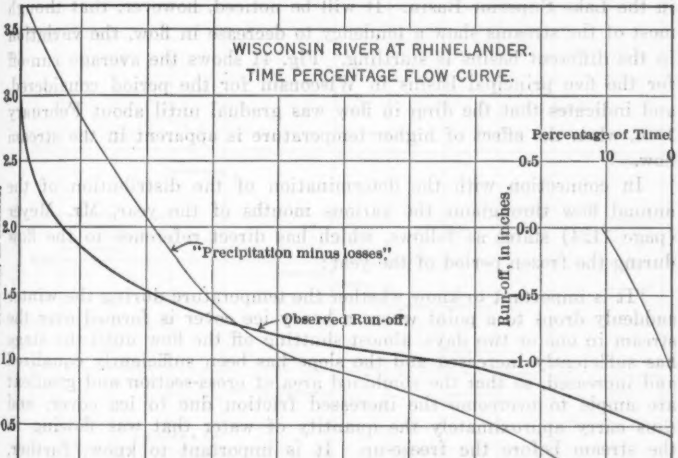

FIG. 40.

losses" would seem to indicate for any given month in a period from September 1st to February 28th or 29th. Apparently, the precipitation during September, October, and part of November, in this section of the country, is used to replenish ground-water lost during the growing period, and the fact that the observed run-off for the period from the middle of November to the latter part of February is less than the run-off computed from "precipitation minus losses," is due entirely to conditions of temperature.

The method which is now followed by the Water Resources Branch of the United States Geological Survey in determining the winter

Mr. Hoyt. flow of streams,* affords a means by which the effect of temperature below freezing can readily be determined for any particular stream.

Table 48 shows the actual run-off, in second-feet per square mile, in 10-day periods, from November 20th, 1914, to March 10th, 1915, for a number of Wisconsin rivers. During a period from the last part of November to the last part of December, the mean temperature dropped from about 30° Fahr. to 0° Fahr. During this same period the mean run-off, in second-feet per square mile, considering all the streams, dropped from 0.56 to about 0.36 sec-ft. per sq. mile, or 36 per cent. The greatest drop for the mean of all the streams was 68% in the Lake Superior Basin. It will be noticed, however, that though most of the streams show a tendency to decrease in flow, the variation in the different basins is startling. Fig. 41 shows the average run-off for the five principal basins in Wisconsin for the period considered, and indicates that the drop in flow was gradual until about February 10th, when the effect of higher temperature is apparent in the stream flow.

In connection with the determination of the distribution of the annual flow throughout the various months of the year, Mr. Meyer (page 1124) states as follows, which has direct reference to the flow during the frozen period of the year:

"It is important to know whether the temperature during the winter suddenly drops to a point where a heavy ice cover is formed over the stream in one or two days, almost shutting off the flow until the stage has sufficiently increased and the slope has been sufficiently equalized and increased, so that the combined area of cross-section and gradient are ample to overcome the increased friction due to ice cover, and thus carry approximately the quantity of water that was flowing in the stream before the freeze-up. It is important to know, further, whether the ground-water table lies so close to the surface that a large portion of the ground-water which would otherwise maintain stream flow during the winter, is frozen up and held until the following spring."

The writer considers it extremely doubtful whether it is possible to determine even approximately the effects of the foregoing conditions without actual measurements. Observation and detailed study at more than seventy-five stations in Minnesota and Wisconsin during the last 3 years have not enabled him to point out any real characteristics as the same year after year. Conditions of temperature differ so radically each winter that there seems to be no method of reasoning by which one can determine in advance, from temperature alone, what the stream flow will be. We know, in a general way, that the flow will drop off materially during cold periods, but there seems to be no relation

* Outlined in Water-Supply Paper 337, "Effects of Ice on Stream Flow."

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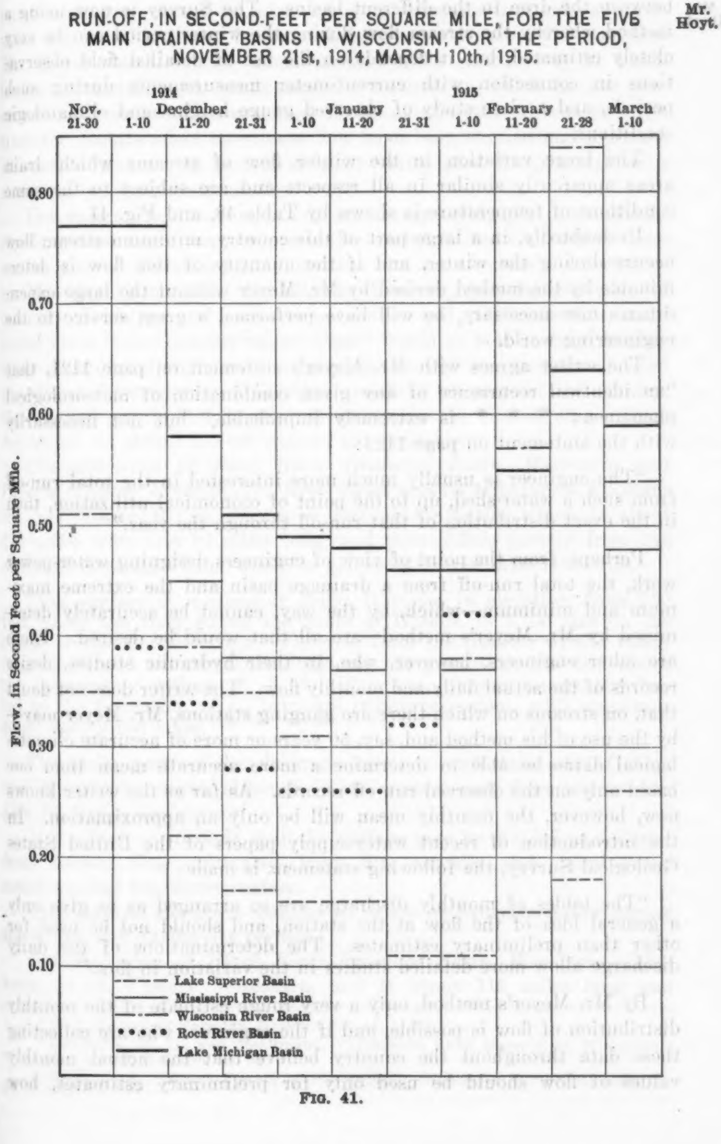

FIG. 41.

Mr. Hoyt. between the drop in the different basins. The Survey is now using a method whereby the stream flow during the winter period can be very closely estimated, but it depends on the use of detailed field observations in connection with current-meter measurements during such periods, and a close study of observed gauge heights and climatologic conditions.

The large variation in the winter flow of streams which drain areas apparently similar in all respects and are subject to the same conditions of temperature is shown by Table 48, and Fig. 41.

Undoubtedly, in a large part of this country, minimum stream flow occurs during the winter, and if the quantity of this flow is determinable by the method devised by Mr. Meyer without the large expenditures now necessary, he will have performed a great service to the engineering world.

The writer agrees with Mr. Meyer's statement on page 1121, that "an identical recurrence of any given combination of meteorological phenomena * * * is extremely improbable," but not necessarily with the statement on page 1122:

"The engineer is usually much more interested in the total run-off from such a water-shed, up to the point of economical utilization, than in the exact distribution of that run-off through the year."

Perhaps, from the point of view of engineers designing water-power work, the total run-off from a drainage basin and the extreme maximum and minimum—which, by the way, cannot be accurately determined by Mr. Meyer's method—are all that would be desired. There are other engineers, however, who, in their hydraulic studies, desire records of the actual daily and monthly flow. The writer does not doubt that, on streams on which there are gauging stations, Mr. Meyer may—by the use of his method and, say, 50 years or more of accurate climatological data—be able to determine a more accurate mean than one based only on the observed run-off records. As far as the writer knows now, however, the monthly mean will be only an approximation. In the introduction of recent water-supply papers of the United States Geological Survey, the following statement is made:

"The tables of monthly discharge are so arranged as to give only a general idea of the flow at the station, and should not be used for other than preliminary estimates. The determinations of the daily discharge allow more detailed studies in the variation in flow."

By Mr. Meyer's method, only a very rough estimate of the monthly distribution of flow is possible, and if the engineers who are collecting these data throughout the country believe that the actual monthly values of flow should be used only for preliminary estimates, how

much more care should be taken in using estimates that only approximate the monthly flow. In general, however, the writer believes that Mr. Meyer should receive much credit for the careful way in which he has undertaken his study and the method by which he has presented his results. The writer is of the opinion that this is the first time the factors affecting stream flow have been studied and analyzed in such a way as to permit of their being used intelligently as a means of estimating stream flow. Mr. Hoyt.

The writer would sum up his discussion by saying that undoubtedly the long-time mean for any particular drainage basin on which there are at present stream-gauging stations can be accurately determined by Mr. Meyer's method. He believes, however, that the estimates of monthly discharge and probable maximum and minimum flow, computed from "precipitation minus losses" would at best be only roughly approximate, and that, until a large number of comparisons of computed and observed run-off have been made in all parts of the country, the method should not be used to determine the run-off of a stream for which no actual run-off records are available.

Characteristics of Rock River Drainage Basin, Wisconsin and Illinois.—Above Rockton, Ill., Rock River and its tributaries, Pecatonica and Sugar Rivers, occupy a rectangular basin, which is about 115 miles wide and 60 miles long, and thus differs greatly from the ordinary drainage basin, which is generally longer than it is wide.

"The surface is moderately hilly; it varies in elevation from 750 feet where the river enters the State of Illinois, to 1 100 feet on the crests of the Kettle Range. The rise from the interior of the valley is gradual, and usually the hilltops are not more than 100 feet above the intervening valleys * * *. This low, uneven topography has led to the formation of an intricate tributary system, with numerous small spring-fed lakes, * * *."*

The main part of the drainage basin is underlain with Galena and Trenton limestone, although in the western part there is considerable St. Peter sandstone.†

It has been estimated‡ that the surface may be divided as follows: 30% forest, 57% cultivated land, 8% swamps and uncultivated meadows, and 5% water surface.

Good natural storage is manifested by the well-sustained flow during the winter and during dry periods.

Characteristics of Black River Drainage Basin, Wisconsin.—This basin is in Western Wisconsin, and is about 115 miles long and

* U. S. Bureau of Forestry, *Bulletin No. 44*, p. 10.

† Hotchkiss, W. O., and Steidtmann, "Limestone Road Material of Wisconsin." Wisconsin Survey, *Bulletin No. 34*.

‡ Smith, L. S., "Water Powers of Wisconsin." Wisconsin Survey, *Bulletin No. 20*, p. 287.

Mr. Hoyt. 20 miles in average width. Above Neillsville, where the gauging station is situated, the length of the basin is about 55 miles, the maximum width about 15 miles, up stream from Neillsville being less than 10 miles, and the maximum width in the upper basin less than 20 miles. Except for a narrow strip under the immediate valley of the river, the basin is underlain with Potsdam sandstone, the remainder being underlain with the older formation, made up of igneous and metamorphic rock. The area above Neillsville is overlain with glacial drift.

The elevation of the basin at the source of the river is somewhat more than 1 400 ft., and the elevation of the river at Neillsville is approximately 985 ft. above sea level. The valleys above Neillsville are V-shaped and narrow. The water reaches the river quickly following precipitation.

Characteristics of Wisconsin River Basin, Above Rhinelander, Wis.—This is a bell-shaped basin, about 42 miles long and 14 miles wide at the Wisconsin-Michigan boundary, gradually increasing to about 40 miles wide at Rhinelander. The soil north of Rhinelander is mostly sand, intermingled with gravel and glacial drift. Little rock outcrops in the entire basin. About 10% of the land is under cultivation or covered with grass; the remainder is second growth and timber. According to A. A. Babcock, Manager of the Wisconsin Valley Improvement Company, approximately 21.10% of the area above Rhinelander is lakes, 0.13% streams, 18.65% swamp, 4.12% kettle holes, and the remainder is high land.

The percentages were computed by Mr. Babcock from the best maps available, many of which were made under his direction. About 45% of the entire area above the gauging station at Rhinelander is under reservoir control.

"The operation [of these reservoirs] throughout the year is about as follows: In the spring of the year as soon as the natural flow of the river below the reservoirs is sufficient to supply the need of the power plants nearest the headwaters, the gates at the outlets of the lakes are closed and water collected. When the summer drought begins the gates are slightly opened and the stored water used to increase the flow in the river. During the fall of the year the natural flow of the river again increases as a result of the fall rains, and the gates are closed and the reservoirs partially refilled. This stored water, and any remaining from the summer period, is then gradually used during the late fall months and winter, the longer period of drought when the precipitation is slight and being stored in the form of snow."*

Mr. Stewart, in the same report (pages 22 and 23), speaking of the drainage basin, says of that portion which is in Vilas, Oneida, and Lincoln Counties:

* Stewart, C. B., "Storage Reservoirs at the Headwaters of the Wisconsin River and Their Relation to Stream Flow," 1911, p. 10.

"The soil of these counties consists of glacial drift of porous sandy material, varying from pure sand in some places to sand mixed with a slight amount of clay in others. * * * The land is more or less rolling but the slopes are gradual with the variations in elevation not exceeding about 100 feet. The most characteristic feature of the topography, especially in Vilas and Oneida Counties, is the sand hills with rounded tops, interspersed with circular or elongated valleys, and occupied by lakes with or without outlets * * *. The character of the soil and topography, together with absence of any erosion indicates that surface flow from rains will not occur except during periods of the early spring when the ground is frozen." Mr. Hoyt.

Characteristics of Wisconsin River Basin, Above Merrill, Wis.—This basin is about 80 miles long and 45 miles wide at its widest part. Conditions as regards geology and soil are much the same as those mentioned for Wisconsin River between Rhinelander and Merrill, Wis. About 580 of the 2 630 sq. miles above Merrill are under control by storage reservoirs, there being about seventeen reservoirs operated by the Wisconsin Valley Improvement Company, with a total capacity of more than 4 000 000 000 cu. ft. Details as to the operation of these reservoirs are given under the heading "Characteristics of Wisconsin River Basin, Above Rhinelander, Wis." According to Mr. A. A. Babcock, this area is made up as follows: 6.39% lakes, 0.30% streams, 21.25% swamps, 3.06% kettle holes, and the remainder high land.

Characteristics of Wisconsin River Basin Between Rhinelander and Merrill, Wis., Including Tomahawk River.—Between Rhinelander and Merrill, Wisconsin River drains an area of triangular shape, about 60 miles long and 45 miles wide at the mouth of Tomahawk River. It is underlain with Pre-Cambrian rock and so deeply covered with drift that rocks outcrop in few places. Of the 1 520 sq. miles which make up this area, about one-third is drained by Tomahawk River and its tributaries. Of this latter area, according to Mr. Babcock, approximately 8.62% is made up of lakes, 0.32% of streams, 24.45% of swamps, 3.61% of kettle holes, and the remainder is high land. Of the total area between Rhinelander and Merrill, from 4 to 6% is under reservoir control. These reservoirs are operated in the same manner as those on Wisconsin River above Rhinelander.

In computing Tables 38 to 48, inclusive, it should be remembered that the writer had to work with the small-scale curves of Figs. 17 and 19, and that undoubtedly there are errors, due to this fact. It is believed that in the final discussion Mr. Meyer should include these two curves on a scale at least twice as large. The writer also used the curves without taking into account abnormal precipitations or low storage conditions, which may change the values somewhat. No attempt has been made to correct final results for changes in ground storage or for seepage flow.

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TABLE 38.—Rock River Below Mouth of Pecatonica River, at Rockton, Ill.

Drainage area, 6 290 sq. miles.

Year and month.	Monthly temperature, in degrees, Fahrenheit.	(a) Monthly precipitation, in inches.	Loss from Land Area. Transpiration. From curve.	Transpiration. Actual.	Evaporation. From curve.	Evaporation. Actual.	Total loss, in inches.	Precipitation minus total loss, in inches.	(b) Observed run-off, in inches.
1903									
November	35	1.18	....	....	0.30	0.31	0.31	0.87	0.50
December	17	1.44	....	....	0.40	0.42	0.42	1.02	0.39
1904									
January	13	0.87	....	....	0.25	0.26	0.26	0.61	0.29
February	12	1.24	....	....	0.30	0.31	0.31	0.93	0.28
March	33	3.62	....	....	1.05	1.10	1.10	2.52	2.62
April	42	2.58	0.21	0.16	1.00	1.05	1.26	1.32	1.92
May	59	3.63	1.70	1.27	1.75	1.84	3.11	0.52	0.99
June	68	1.89	2.30	1.72	1.22	1.28	3.00	—1.11	0.45
July	71	3.39	2.50	1.87	2.00	2.10	3.97	—0.58	0.30
August	69	4.34	2.20	1.65	2.15	2.26	3.91	0.43	0.28
September	63	5.75	1.70	1.27	2.45	2.57	3.84	1.91	0.43
October	53	2.27	0.88	0.66	1.00	1.05	1.66	0.61	0.57
	44.6	32.20	11.49	8.60	13.87	14.55	23.15	9.05	9.02
1904									
November	42	0.27	....	....	0.20	0.21	0.21	0.06	0.34
December	23	2.56	....	....	0.45	0.47	0.47	2.09	0.34
1905									
January	12	1.12	....	....	0.30	0.31	0.31	0.81	0.40
February	13	1.54	....	....	0.30	0.31	0.31	1.23	0.80
March	36	2.90	....	....	0.97	1.03	1.03	1.87	2.33
April	47	2.52	0.75	0.56	1.10	1.16	1.72	0.80	1.80
May	57	6.22	1.52	1.14	2.42	2.54	3.68	2.54	1.23
June	68	4.12	2.30	1.73	2.20	2.31	4.04	0.08	1.15
July	71	3.52	2.50	1.87	2.00	2.10	3.97	—0.45	0.73
August	73	4.25	2.52	1.89	2.30	2.42	4.31	—0.06	0.48
September	67	1.60	2.05	1.54	1.00	1.05	2.59	—0.99	0.48
October	51	3.46	0.70	0.52	1.30	1.37	1.89	1.57	0.47
	46.6	34.08	12.31	9.25	14.54	15.28	24.58	9.55	10.05

(a) Taken from mean of nine U. S. Weather Bureau precipitation stations in or adjacent to Rock River Drainage Basin above Rockton, Ill.

(b) Water Resources of Illinois, Report of Rivers and Lakes Commission of Illinois, by U. S. Geological Survey, 1914.

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TABLE 38.—(*Continued.*)

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area. Transpiration. From curve.	Transpiration. Actual.	Evaporation. From curve.	Evaporation. Actual.	Total loss, in inches.	Precipitation minus total loss, in inches.	Observed run-off, in inches.
1905									
November......	38	2.42	....	....	0.60	0.63	0.63	1.79	0.45
December......	28	1.44	....	....	0.25	0.26	0.26	1.18	0.51
1906									
January........	26	2.90	....	....	0.48	0.50	0.50	2.40	1.76
February.......	22	1.52	....	....	0.50	0.52	0.52	1.00	1.64
March..........	27	2.34	....	....	0.72	0.76	0.76	1.58	2.29
April...........	51	1.49	1.10	0.83	0.80	0.84	1.67	—0.18	1.63
May............	58	3.14	1.60	1.20	1.60	1.68	2.88	0.26	0.67
June...........	67	4.05	2.22	1.66	2.15	2.26	3.92	0.13	0.42
July............	72	2.11	2.60	1.95	1.40	1.47	3.42	—1.31	0.28
August.........	74	6.36	2.61	1.95	3.15	3.31	5.26	1.10	(a) 0.35
September......	69	3.21	2.20	1.64	1.80	1.81	3.45	—0.24	(a) 0.20
October........	52	2.55	0.78	0.58	1.00	1.05	1.63	0.92	(a) 0.30
	48.6	33.53	13.11	9.81	13.91	15.09	24.90	8.63	10.50
1906									
November......	38	2.76	....	....	0.71	0.75	0.75	2.01	0.47
December......	27	1.52	....	....	0.25	0.26	0.26	1.26	0.61
1907									
January........	19	2.94	....	....	0.58	0.61	0.61	2.33	1.15
February.......	23	0.43	....	....	0.50	0.52	0.52	—0.09	(a) 0.80
March..........	40	2.16	....	....	0.90	0.95	0.95	1.21	0.88
April...........	40	3.22	....	....	1.15	1.21	1.21	2.01	1.32
May............	51	3.17	1.1	0.82	1.40	1.47	2.29	0.88	0.74
June...........	67	4.80	2.2	1.65	2.40	2.52	4.17	0.63	0.93
July............	73	6.59	2.6	1.95	3.25	3.41	5.36	1.23	1.09
August.........	69	3.92	2.2	1.65	2.00	2.10	3.75	0.17	0.73
September.....	62	6.05	1.6	1.20	2.52	2.64	3.84	2.21	0.80
October........	49	1.09	0.5	0.38	0.42	0.44	0.82	0.27	0.82
	46.5	38.65	10.2	7.65	16.08	16.88	24.53	14.12	10.34
1907									
November......	38	1.19	....	....	0.35	0.37	0.37	0.82	0.82
December......	29	1.55	....	....	0.25	0.26	0.26	1.29	0.36
1908									
January........	26	1.21	....	....	0.62	0.65	0.65	0.56	0.43
February.......	25	1.85	....	....	0.66	0.69	0.69	1.16	1.13
March..........	38	2.73	....	....	0.97	1.02	1.02	1.71	1.99
April...........	52	4.17	1.18	0.89	1.75	1.84	2.73	1.44	1.19
May............	61	5.92	1.80	1.35	2.50	2.62	3.97	1.95	1.67
June...........	67	3.48	2.22	1.66	1.90	2.00	3.66	—0.18	1.14
July............	73	2.59	2.68	2.01	1.65	1.73	3.74	—1.15	0.65
August.........	70	3.52	2.30	1.72	1.89	1.97	3.69	—0.17	0.33
September.....	68	1.51	2.12	1.59	0.95	1.00	2.59	—1.08	0.26
October........	52	1.24	0.78	0.59	0.55	0.58	1.17	0.07	0.28
	49.9	30.96	13.08	9.81	14.04	14.73	24.54	6.42	10.25

(a) Estimated.

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TABLE 39.—SUMMARY OF DATA AND COMPUTATIONS FOR ROCK RIVER DRAINAGE BASIN ABOVE ROCKTON, ILL.

Drainage area, 6 290 sq. miles.

Year.	(a) Rainfall.	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	(a) Observed run-off.
1904	32.20	14.55	8.60	23.15	9.05	9.02
1905	34.08	15.28	9.25	24.53	9.55	10.05
1906	33.53	15.09	9.81	24.90	8.63	(b) 10.50
1907	38.65	16.88	7.65	24.53	14.12	10.34
1908	30.96	14.73	9.81	24.54	6.42	10.25
Total....	169.42	76.53	45.12	121.65	47.77	50.16
Mean...	33.88	15.31	9.02	24.33	9.55	10.03

(a) November 1st of previous year to October 31st of given year.
(b) Run-off estimated from August 1st to October 31st.

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TABLE 40.—DATA AND COMPUTATIONS FOR BLACK RIVER AT NEILLSVILLE, WIS.

Drainage area, 774 sq. miles.

Year and month.	Monthly temperature, in degrees, Fahrenheit.	(a) Monthly precipitation, in inches.	Loss from Land Area. Transpiration. From curve.	Loss from Land Area. Transpiration. Actual.	Loss from Land Area. Evaporation. From curve.	Loss from Land Area. Evaporation. Actual.	Total loss, in inches.	Precipitation, minus total loss, in inches.	(b) Observed run-off, in inches.
1905									
November......	32	1.25	....	....	0.22	0.19	0.19	1.06	0.56
December......	22	0.80	....	....	0.20	0.18	0.18	0.62	0.43
1906									
January........	20	2.01	....	....	0.60	0.53	0.53	1.48	(c) 0.20
February.......	16	0.54	....	....	0.40	0.35	0.35	0.19	(c) 0.20
March..........	22	1.91	....	....	0.68	0.60	0.60	1.31	(c) 0.50
April..........	47	1.39	0.72	0.50	0.62	0.55	1.05	0.34	5.57
May............	55	5.82	1.40	0.98	2.28	2.00	2.98	2.84	2.16
June...........	63	4.23	1.95	1.36	2.10	1.85	3.21	1.02	1.05
July...........	68	2.85	2.30	1.61	1.70	1.49	3.10	—0.25	0.27
August.........	69	3.40	2.20	1.54	1.82	1.60	3.14	0.26	0.28
September......	64	3.40	1.80	1.26	1.65	1.45	2.71	0.69	0.40
October........	47	3.02	0.35	0.24	1.05	0.92	1.16	1.86	0.44
	43.7	30.62	10.72	7.49	13.32	11.71	19.20	11.42	12.06
1906									
November......	32	2.70	....	....	0.52	0.46	0.46	2.24	1.06
December......	21	1.24	....	....	0.32	0.28	0.28	0.96	0.80
1907									
January........	13	1.57	....	....	0.28	0.25	0.25	1.32	(c) 0.20
February.......	17	0.65	....	....	0.44	0.39	0.39	0.26	(c) 0.20
March..........	35	1.82	....	....	0.78	0.69	0.69	1.13	3.11
April..........	36	1.34	....	....	0.68	0.60	0.60	0.74	1.84
May............	47	2.89	0.72	0.50	1.20	1.05	1.55	1.34	1.05
June...........	63	3.61	1.95	1.36	1.88	1.65	3.01	0.60	0.26
July...........	69	2.69	2.20	1.54	1.55	1.36	2.90	—0.21	0.73
August.........	64	4.73	1.80	1.26	2.12	1.86	3.12	1.61	0.14
September......	57	4.16	1.20	0.84	1.72	1.51	2.35	1.81	0.68
October........	44	1.02	0.10	0.07	0.40	0.35	0.42	0.60	0.15
	41.5	28.42	7.97	5.57	11.89	10.45	16.02	12.40	10.22

(a) Mean of five U. S. Weather Bureau precipitation stations in or adjacent to Black River Drainage Basin above Neillsville.

(b) Run-off by U. S. Geological Survey. Figures revised on account of error of drainage basin as published in Water Supply Papers 171, 207, 245, and 265.

(c) Estimated.

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TABLE 40.—(*Continued.*)

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area. Transpiration. From curve.	Loss from Land Area. Transpiration. Actual.	Loss from Land Area. Evaporation. From curve.	Loss from Land Area. Evaporation. Actual	Total loss, in inches.	Precipitation minus total loss, in inches.	Observed run-off, in inches.
1907									
November......	32	0.94	....	....	0.20	0.18	0.18	0.76	0.11
December......	23	0.57	....	....	0.20	0.18	0.18	0.39	0.08
1908									
January........	18	0.65	....	....	0.42	0.37	0.37	0.28	0.10
February.......	17	1.37	....	....	0.45	0.40	0.40	0.97	0.10
March..........	32	2.31	....	....	0.82	0.72	0.72	1.59	1.00
April...........	44	3.29	0.42	0.29	1.25	1.10	1.39	1.90	3.31
May............	55	5.21	1.40	0.97	2.10	1.85	2.82	2.39	2.92
June...........	63	6.10	1.95	1.36	2.65	2.33	3.69	2.41	2.08
July............	69	3.92	2.36	1.65	2.15	1.89	3.54	0.38	1.26
August.........	66	1.92	1.98	1.39	1.10	0.96	2.35	—0.43	0.08
September......	64	2.98	1.80	1.26	1.50	1.32	2.58	0.40	0.05
October.........	48	2.09	0.42	0.29	0.80	0.70	0.99	1.10	0.17
	44.2	31.35	10.33	7.21	13.64	12.00	19.21	12.14	11.21
1908									
November......	34	1.50	....	....	0.32	0.28	0.28	1.22	0.26
December......	19	1.28	....	....	0.40	0.35	0.35	0.93	0.17
1909									
January........	16	0.77	....	....	0.32	0.28	0.28	0.49	0.15
February.......	20	1.95	....	....	0.56	0.49	0.49	1.46	0.07
March..........	27	1.40	....	....	0.62	0.55	0.55	0.85	0.21
April...........	38	3.40	....	....	1.12	0.98	0.98	2.42	No
May............	53	2.92	1.25	0.88	1.35	1.19	2.07	0.85	run-off
June...........	65	4.22	2.08	1.45	2.15	1.89	3.34	0.88	data,
July............	68	2.55	2.30	1.61	1.52	1.34	2.95	—0.40	Apr.,
August.........	70	2.88	2.30	1.61	1.67	1.47	3.08	—0.20	1909,
September......	58	4.46	1.29	0.90	1.85	1.63	2.53	1.93	to
October.........	44	2.09	0.10	0.07	0.66	0.58	0.65	1.45	Nov., 1913.
	42.6	29.42	9.32	6.52	12.54	11.03	17.55	11.87	......

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TABLE 40.—(*Continued.*)

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	LOSS FROM LAND AREA. Transpiration. From curve.	Transpiration. Actual.	Evaporation. From curve.	Evaporation. Actual.	Total loss, in inches.	Precipitation minus total loss, in inches.	Observed run-off, in inches.
1909									
November......	39	5.10	....	....	1.21	1.06	1.06	4.04	
December......	11	1.41	....	....	0.21	0.18	0.18	1.23	....
1910									
January.........	18	1.44	....	....	0.32	0.28	0.28	1.16	....
February........	12	0.30	....	....	0.28	0.25	0.25	0.05	....
March...........	43	0.04	....	....	0.31	0.27	0.27	−0.23	....
April............	47	2.84	0.75	0.52	1.17	1.03	1.55	1.29	....
May.............	51	3.05	1.08	0.76	1.40	1.23	1.99	1.06	....
June............	67	0.59	1.22	0.50	0.50	0.44	1.29	−0.70	....
July.............	72	2.61	2.60	1.82	1.65	1.45	3.27	−0.66	....
August..........	68	4.34	2.12	1.48	2.10	1.85	3.33	1.01	....
September......	58	2.35	1.29	0.90	1.08	0.95	1.85	0.50	....
October	51	1.60	0.69	0.48	0.68	0.60	1.08	0.52	....
	44.3	25.67	9.75	6.81	10.91	9.59	16.40	9.27	
1910									
November......	26	0.69	....	....	0.15	0.13	0.13	0.56	
December......	17	0.77	....	....	0.20	0.17	0.17	0.60	....
1911									
January.........	15	1.02	....	....	0.37	0.32	0.32	0.70	....
February........	22	1.24	....	....	0.48	0.42	0.42	0.82	....
March...........	33	1.36	....	....	0.70	0.62	0.62	0.74	....
April............	42	1.11	0.22	0.15	0.62	0.55	0.70	0.41	....
May.............	61	5.82	1.82	1.27	2.50	2.20	3.47	2.35	....
June............	69	5.12	2.38	1.66	2.60	2.28	3.94	1.18	....
July.............	71	4.94	2.50	1.75	2.60	2.28	4.03	0.91	....
August..........	66	2.94	1.97	1.37	1.52	1.34	2.71	0.23	....
September......	59	6.92	1.38	0.97	2.60	2.28	3.25	3.67	....
October.........	45	9.37	0.18	0.12	2.25	1.98	2.10	7.27	....
	43.7	41.30	10.51	7.29	16.59	14.57	21.86	19.44	....

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TABLE 40.—(*Continued.*)

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area. Transpiration. From curve.	Transpiration. Actual.	Evaporation. From curve.	Evaporation. Actual.	Total loss, in inches.	Precipitation minus total loss, in inches.	Observed run-off, in inches.
1911									
November......	24	1.95	....	....	0.36	0.32	0.32	1.63	....
December......	24	2.52	....	....	0.45	0.40	0.40	2.12	....
1912									....
January........	−6	0.57	....	....	0.05	0.04	0.04	0.53	....
February.......	11	0.13	....	....	0.25	0.22	0.22	−0.09	....
March..........	22	0.50	....	....	0.48	0.42	0.42	0.08	....
April...........	47	3.14	0.72	0.51	1.30	1.14	1.65	1.49	....
May.............	57	5.85	1.52	1.06	2.32	2.04	3.10	2.75	....
June............	65	1.04	2.09	1.46	0.82	0.72	2.18	−1.14	....
July............	69	6.76	2.36	1.65	2.80	2.46	4.11	2.65	....
August.........	65	8.08	1.87	1.31	3.25	2.85	4.16	3.92	....
September.....	61	4.01	1.53	1.07	1.80	1.57	2.64	1.87	....
October........	49	1.80	0.52	0.36	0.70	0.62	0.98	0.82	....
	40.6	36.35	10.12	7.42	14.58	12.80	20.22	16.13	....
1912									
November......	35	0.83	....	....	0.20	0.18	0.18	0.65	....
December......	23	2.27	....	....	0.42	0.37	0.37	1.90	....
1913									....
January........	15	0.58	....	....	0.31	0.27	0.27	0.31	....
February.......	10	0.93	....	....	0.20	0.18	0.18	0.75	....
March..........	25	3.18	....	....	0.80	0.70	0.70	2.48	....
April...........	47	2.39	0.74	0.52	1.00	0.88	1.40	0.99	....
May.............	55	5.63	1.40	0.97	2.20	1.94	2.91	2.72	....
June............	70	2.47	2.42	1.69	1.52	1.34	3.03	−0.56	....
July............	69	6.04	2.36	1.65	2.95	2.60	4.25	1.79	....
August.........	70	2.26	2.30	1.61	1.82	1.16	2.77	−0.51	....
September.....	61	3.19	1.54	1.08	1.50	1.32	2.40	0.79	....
October........	38	1.94	....	....	0.46	0.40	0.40	1.54	....
	43.2	31.71	10.76	7.52	12.88	11.34	18.86	12.85	....
1913									
November......	38	1.51	....	....	0.40	0.35	0.35	1.18	(c) 0.30
December......	28	1.12	....	....	0.12	0.06	0.06	0.06	(c) 0.10
1914									
January........	22	1.95	....	....	0.60	0.53	0.53	1.42	0.18
February.......	7	0.51	....	....	0.14	0.12	0.12	0.39	0.25
March..........	29	1.59	....	....	0.70	0.62	0.62	0.97	1.38
April...........	43	3.08	0.32	0.22	1.20	1.06	1.28	1.80	2.87
May.............	60	4.48	1.76	1.23	2.08	1.83	3.06	1.42	1.94
June............	66	9.21	2.15	1.51	4.05	3.56	5.07	4.14	4.11
July............	75	2.40	2.92	2.04	1.60	1.41	3.45	−1.05	0.76
August.........	69	4.48	2.20	1.55	2.23	1.96	3.51	0.97	0.28
September.....	60	4.61	1.45	1.02	1.90	1.67	2.69	1.92	1.29
October........	53	2.06	0.85	0.59	1.92	1.68	2.27	−0.21	0.69
	45.7	36.00	11.65	8.15	16.95	14.85	23.01	12.99	14.15

(c) Partly estimated.

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TABLE 41.—Summary of Data and Computations for Black River Basin, Above Neillsville, Wis.

Drainage area, 774 sq. miles.

Year.	(a) Rainfall.	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	(b) Observed run-off
1906	30.62	11.71	7.49	19.20	11 42	12.93
1907	28.42	10.45	5.57	16.02	12.40	8.35
1908	31.35	12.00	7.21	19.21	12. 4	11.49
1909	29.42	11.03	6.52	17.55	11.87	......
1910	25.67	9.59	6.81	16.40	9.27	......
1911	41.30	14.57	7.29	21.86	19.44	......
1912	(c)36.35	12.80	7.42	20.22	16.13	......
1913	31.71	11.34	7.52	18.86	12.85	......
1914	36.00	14.85	8.15	23.07	12.99	13.80
Total...	290.84	108.34	64.68	173.09	118.51	(d) 46.57
Mean...	32.32	12.04	7.19	19.23	13.18	(d) 11.64

(a) November 1st of previous year to October 31st of given year.

(b) Run-off, from March 1st of given year to February 28th or 29th of following year.

(c) 35.11 in. from May to October

(d) For partial period.

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TABLE 42.—DATA AND COMPUTATIONS FOR WISCONSIN RIVER BASIN, ABOVE RHINELANDER, WIS.

Drainage area, 1 110 sq. miles.

Year and month.	Monthly temperature, in degrees, Fahrenheit.	(a) Monthly precipitation, in inches.	Evaporation from Water Areas.		Loss from Land Areas.				Total loss, in inches.	Precipitation minus total loss, in inches.	(b) Observed run-off, in inches.
					Transpiration.		Evaporation.				
			From curve.	Actual.	From curve.	Actual.	From curve.	Actual.			
1908											
November..	32.6	1.37	1.16	0.12	....	....	0.25	0.13	0.25	1.12	0.42
December..	17.4	0.88	0.40	0.04	....	....	0.20	0.11	0.15	0.73	0.78
1909											
January....	15.4	0.52	0.32	0.03	....	....	0.30	0.16	0.19	0.23	0.91
February ..	17.6	1.15	0.51	0.05	....	....	0.40	0.21	0.26	0.89	0.77
March......	25.0	1.09	1.00	0.10	....	....	0.60	0.32	0.42	0.67	0.67
April.......	34.0	3.33	1.80	0.18	....	....	1.00	0.53	0.71	2.62	1.04
May........	51.7	1.90	2.32	0.23	1.15	0.71	0.95	0.50	1.44	0.46	1.94
June........	65.2	3.10	3.68	0.37	2.10	1.30	1.72	0.91	2.58	0.52	1.26
July.........	66.8	6.56	4.08	0.41	2.22	1.38	3.10	1.64	3.43	3.13	1.29
August.....	67.8	2.93	4.18	0.42	2.30	1.43	1.58	0.84	2.69	0.24	1.71
September..	57.4	2.01	3.15	0.32	1.20	0.75	0.98	0.52	1.59	0.42	1.09
October....	43.1	1.22	1.92	0.19	....	....	0.40	0.21	0.40	0.82	0.83
	41.1	26.06	24.52	2.46	8.97	5.57	11.48	6.08	14.11	11.95	12.71
1909											
November..	37.0	5.10	1.48	0.15	....	....	1.12	0.59	0.74	4.36	1.53
December..	16.4	0.99	0.40	0.04	....	....	0.28	0.15	0.19	0.80	1.11
1910											
January....	18.8	0.81	0.28	0.03	....	....	0.30	0.16	0.19	0.62	1.21
February ..	11.4	1.20	0.28	0.03	....	....	0.25	0.13	0.16	1.04	1.23
March......	40.0	0.38	2.50	0.25	....	....	0.42	0.22	0.47	—0.09	1.18
April.......	46.8	2.01	1.92	0.19	0.72	0.45	0.92	0.49	1.13	0.88	1.19
May........	49.2	2.54	2.11	0.21	0.92	0.57	1.18	0.63	1.41	1.13	1.03
June........	65.7	0.34	3.72	0.37	1.15	0.71	0.40	0.21	1.29	—0.95	0.60
July.........	69.2	1.91	4.16	0.42	2.35	1.45	1.28	0.68	2.55	—0.64	0.56
August.....	66.2	3.14	4.05	0.40	1.95	1.21	1.68	0.89	2.50	0.64	0.53
September..	52.6	2.32	3.02	0.30	0.88	0.55	0.92	0.49	1.34	0.98	0.60
October....	48.6	1.95	2.38	0.24	0.52	0.32	0.60	0.32	0.88	1.07	0.56
	43.2	22.69	26.30	2.63	8.49	5.26	9.38	4.96	12.85	9.84	11.33

(a) Mean determined from seven U. S. Weather Bureau stations, in and adjacent to the Wisconsin Basin above Rhinelander.

(b) As determined by the U. S. Geological Survey.

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TABLE 42.—(*Continued.*)

Year and month.	Monthly temperature, in degrees. Fahrenheit.	Monthly precipitation, in inches.	Evaporation from Water Areas.		Loss from Land Areas.				Total loss, in inches.	Precipitation minus total loss, in inches.	Observed run-off, in inches.
					Transpiration.		Evaporation.				
			From curve.	Actual.	From curve.	Actual.	From curve.	Actual.			
1910											
November..	25.7	1.05	0.78	0.08	....	....	0.20	0.11	0.19	0.86	0.60
December..	15.9	0.71	0.35	0.04	....	....	0.30	0.16	0.20	0.51	0.56
1911											
January ...	12.4	0.73	0.28	0.03	....	....	0.30	0.16	0.19	0.54	0.84
February ..	20.4	1.08	0.68	0.07	....	....	0.52	0.28	0.35	0.73	0.75
March......	30.2	1.65	1.42	0.14	....	....	0.60	0.32	0.46	1.19	0.97
April.......	42.0	0.64	1.60	0.16	0.22	0.14	0.50	0.26	0.56	0.08	1.12
May	59.4	5.10	3.05	0.30	1.68	1.04	2.20	1.17	2.51	2.59	1.08
June	69.6	1.32	4.18	0.42	2.42	1.50	1.00	0.53	2.45	—1.13	0.72
July	67.8	7.43	3.96	0.40	2.30	1.42	3.95	2.10	3.92	3.51	1.16
August.....	68.8	5.22	3.79	0.38	1.80	1.12	2.32	1.23	2.73	2.49	1.84
September.	56.6	3.04	3.07	0.31	1.20	0.75	1.40	0.74	1.80	1.24	1.45
October	48.4	6.25	1.95	0.20	....	....	1.70	0.90	1.10	5.15	2.79
	42.3	34.22	25.11	2.53	9.62	5.97	14.99	7.96	16.46	17.76	13.83
1911											
November..	24.4	2.82	0.68	0.07	....	....	0.48	0.25	0.32	2.50	1.02
December..	22.9	2.57	1.90	0.19	....	....	0.30	0.16	0.35	2.22	1.43
1912											
January....	—5.4	0.63	0.01	0.00	....	....	0.10	0.05	0.05	0.58	1.04
February ..	11.4	0.21	0.28	0.03	....	....	0.25	0.13	0.16	0.05	0.97
March......	19.8	0.42	0.64	0.06	....	....	0.48	0.25	0.31	0.11	1.04
April.......	42.1	2.76	1.62	0.16	0.22	0.14	1.00	0.53	0.83	1.93	2.21
May	55.2	4.31	2.55	0.26	1.40	0.87	1.85	0.98	2.11	2.20	1.70
June	61.6	2.16	3.29	0.33	1.88	1.17	1.30	0.69	2.19	—0.03	1.33
July	67.1	3.68	3.92	0.40	2.22	1.37	2.00	1.06	2.83	0.85	1.37
August.....	61.8	7.68	3.60	0.37	1.62	1.01	2.95	1.56	2.94	4.74	3.07
September.	58.4	2.76	3.22	0.32	1.28	0.79	1.28	0.68	1.79	0.97	2.43
October	48.0	1.92	2.32	0.23	0.44	0.27	0.70	0.37	0.87	1.05	1.75
	38.9	31.92	24.03	2.42	9.06	5.62	12.69	6.71	14.75	17.12	19.36

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TABLE 42.—(*Continued.*)

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Evaporation from Water Areas.		Loss from Land Areas.				Total loss, in inches.	Precipitation minus total loss, in inches.	Observed run-off, in inches.
					Transpiration.		Evaporation.				
			From curve.	Actual.	From curve.	Actual.	From curve.	Actual.			
1912											
November..	33.1	0.49	1.21	0.12	....	....	0.15	0.08	0.20	0.29	1.76
December..	21.6	2.07	0.56	0.06	....	....	0.42	0.22	0.28	1.79	1.53
1913											
January....	14.0	0.53	0.30	0.03	....	....	0.25	0.13	0.16	0.37	0.77
February ..	8.4	0.96	0.20	0.02	....	....	0.18	0.10	0.12	0.84	0.70
March	21.6	2.47	0.72	0.07	....	....	0.65	0.34	0.41	2.06	1.18
April.......	43.8	1.70	1.80	0.18	0.42	0.26	0.80	0.42	0.86	0.84	2.11
May	52.0	3.68	2.35	0.24	1.08	0.67	1.60	0.85	1.76	1.92	1.65
June	65.8	3.35	3.78	0.38	2.15	1.33	1.88	1.00	2.71	0.64	1.27
July........	65.6	5.92	3.76	0.38	2.15	1.33	2.70	1.43	3.14	2.78	1.37
August.....	66.4	3.14	4.02	0.40	1.97	1.22	1.70	0.90	2.62	0.52	1.04
September..	56.9	3.81	3.12	0.31	1.20	0.75	1.58	0.84	1.90	1.91	1.18
October....	44.7	3.31	2.02	0.20	0.10	0.06	1.08	0.57	0.83	2.48	1.74
	41.1	31.43	23.84	2.39	9.07	5.62	12.69	6.88	14.99	16.64	16.30
1913											
November..	36.8	1.51	1.45	0.14	....	....	0.40	0.21	0.35	1.16	1.58
December..	28.3	0.15	0.90	0.09	....	....	0.22	0.12	0.21	—0.06	1.27
1914											
January....	19.8	1.44	0.48	0.05	....	....	0.35	0.19	0.24	1.20	1.05
February ..	6.6	0.40	0.16	0.02	....	....	0.18	0.10	0.12	0.28	0.95
March......	24.5	1.66	0.95	0.09	....	....	0.62	0.33	0.42	1.24	1.08
April.......	38.5	3.44	1.40	0.14	....	....	1.15	0.61	0.75	2.69	1.25
May........	56.5	2.18	3.08	0.31	1.48	0.92	1.15	0.61	1.84	0.34	1.27
June........	61.8	5.62	3.34	0.33	1.88	1.16	2.45	1.30	2.79	2.83	1.15
July........	71.1	5.09	4.45	0.44	2.50	1.55	2.72	1.44	3.43	1.66	1.96
August.....	64.6	6.11	3.82	0.38	1.89	1.17	2.60	1.38	2.93	3.18	1.89
September.	58.2	2.62	3.25	0.32	1.30	0.81	1.20	0.64	1.77	0.85	1.47
October....	52.8	0.98	2.78	0.28	0.85	0.53	0.48	0.25	1.06	—0.08	1.02
	43.2	31.20	26.06	2.59	9.90	6.14	13.52	7.18	15.91	15.29	16.38

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TABLE 43.—SUMMARY OF DATA AND COMPUTATIONS FOR WISCONSIN RIVER BASIN, ABOVE RHINELANDER, WIS.

Drainage area, 1 110 sq. miles.

Year.	(a) Rainfall.	EVAPORATION.		Transpiration.	Total loss.	Precipitation minus total loss.	(b) Observed run-off.
		Water.	Land.				
1909	26.06	2.46	6.08	5.57	14.11	11.95	15.42
1910	22.69	2.63	4.96	5.26	12.85	9.84	8.79
1911	34.22	2.53	7.96	5.97	16.46	17.76	15.61
1912	31.92	2.42	6.71	5.62	14.75	17.12	19.80
1913	31.43	2.39	6.88	5.62	14.99	16.64	16.29
1914	31.20	2.59	7.18	6.14	15.91	15.29	14.69
Total... ...	177.52	15.02	39.77	34.18	89.07	88.60	90.60
Average....	29.60	2.50	6.63	5.70	14.83	14.77	15.10

(a) Precipitation, November 1st of preceding year to October 31st of given year.

(b) Run-off, March 1st of given year to February 28th or 29th of following year.

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TABLE 44.—DATA AND COMPUTATIONS FOR WISCONSIN RIVER BASIN ABOVE MERRILL, WIS.

Drainage area, 2 630 sq. miles.

Year and month.	Monthly temperature, in degrees, Fahrenheit.	(a) Monthly precipitation, in inches.	Loss from Land Area. Transpiration. From curve.	Transpiration. Actual.	Evaporation. From curve.	Evaporation. Actual.	Total loss, in inches.	Precipitation minus total loss, in inches.	(b) Observed run-off, in inches.
1908									
November.....	32.6	1.47	....	....	0.25	0.17	0.17	1.30	0.57
December.....	17.4	1.09	....	....	0.20	0.13	0.13	0.96	0.65
1909									
January.......	15.4	0.53	....	....	0.42	0.28	0.28	0.25	0.65
February......	17.6	1.26	....	....	0.50	0.33	0.33	0.93	0.62
March........	25.0	1.29	....	....	0.62	0.41	0.41	0.88	0.57
April	34.0	2.95	....	....	0.92	0.61	0.61	2.34	1.92
May	51.7	2.11	1.15	0.76	1.00	0.66	1.43	0.68	3.14
June...........	65.2	3.36	2.10	1.39	1.90	1.25	2.64	0.72	1.50
July...........	66.8	5.26	2.22	1.47	2.60	1.72	3.19	2.07	1.06
August........	67.8	2.41	2.30	1.52	1.45	0.96	2.48	—0.07	0.81
September	57.4	2.45	1.20	0.79	1.15	0.76	1.55	0.90	0.70
October........	43.1	1.71	....	....	0.52	0.34	0.34	1.37	0.62
	41.1	25.89	8.97	5.93	11.53	7.62	13.55	12.34	12.81
1909									
November.....	37.0	4.48	....	....	1.05	0.69	0.69	3.79	1.80
December......	16.4	0.93	....	....	0.35	0.23	0.23	0.70	1.04
1910									
January.......	13.8	0.76	....	....	0.38	0.25	0.25	0.51	0.92
February......	11.4	0.97	....	....	0.25	0.17	0.17	0.80	0.82
March..........	40.0	0.29	....	....	0.42	0.28	0.28	0.01	1.29
April	46.8	2.46	0.72	0.48	1.10	0.73	1.21	1.25	1.75
May............	49.2	2.42	0.92	0.61	1.15	0.76	1.37	1.05	1.21
June...........	65.7	0.38	1.15	0.76	0.45	0.30	1.06	—0.68	0.53
July...........	69.2	2.13	2.85	1.55	1.45	0.96	2.51	—0.38	0.41
August........	66.2	3.06	1.95	1.29	1.60	1.06	2.35	0.71	0.42
September.....	56.2	2.40	0.88	0.58	1.09	0.72	1.30	1.10	0.54
October........	48.6	2.12	0.52	0.34	0.88	0.58	0.92	1.20	0.63
	43.4	22.40	8.49	5.61	10.17	6.73	12.34	10.06	11.36

(a) Precipitation from mean of thirteen U. S. Weather Bureau stations in and adjacent to the Wisconsin River water-shed above Merrill, Wis.

(b) Run-off by U. S. Geological Survey.

Mr. Hoyt.

TABLE 44.—(*Continued.*)

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area. Transpiration. From curve.	Loss from Land Area. Transpiration. Actual.	Loss from Land Area. Evaporation. From curve.	Loss from Land Area. Evaporation. Actual.	Total loss, in inches.	Precipitation minus total loss, in inches.	Observed run-off, in inches.
1910									
November	25.7	1.12	....	....	0.22	0.15	0.15	0.97	0.50
December	15.9	0.83	....	....	0.32	0.21	0.21	0.62	0.53
1911									
January	12.4	0.76	....	....	0.30	0.20	0.20	0.56	0.62
February	20.4	1.12	....	....	0.52	0.34	0.34	0.78	0.58
March	30.2	1.69	....	....	0.70	0.46	0.46	1.23	1.12
April	42.0	0.78	0.22	0.15	0.52	0.34	0.49	0.29	1.66
May	59.4	4.97	1.68	1.11	2.18	1.48	2.59	2.38	1.56
June	69.6	1.51	2.42	1.60	1.10	0.72	2.32	—0.81	0.81
July	67.8	7.05	2.30	1.52	3.30	2.18	3.70	3.35	0.64
August	63.8	4.72	1.80	1.19	2.11	1.39	2.58	2.14	1.00
September	56.6	4.17	1.20	0.79	1.68	1.11	1.90	2.27	1.29
October	43.4	6.52	....	....	1.70	1.12	1.12	5.40	3.79
	42.3	35.24	9.62	6.36	14.65	9.70	16.06	19.18	14.10
1911									
November	24.4	2.77	....	....	0.45	0.30	0.30	2.47	1.42
December	22.9	2.52	....	....	0.40	0.26	0.26	2.26	(1.80) a
1912									
January	— 5.4	0.56	....	....	0.05	0.03	0.03	0.53	(1.20) a
February	11.4	0.27	....	....	0.28	0.18	0.18	0.09	(1.00) a
March	19.8	0.37	....	....	0.42	0.28	0.28	0.09	(1.00) a
April	42.1	3.06	0.22	0.15	1.15	0.76	0.91	2.15	3.05
May	55.2	4.89	1.40	0.92	2.00	1.32	2.24	2.65	2.78
June	61.6	1.78	1.88	1.24	1.08	0.71	1.95	—0.17	1.30
July	67.1	5.44	2.22	1.47	2.62	1.73	3.20	2.24	1.60
August	61.8	7.84	1.62	1.07	2.95	1.95	3.02	4.82	2.39
September	58.4	2.94	1.28	0.84	1.30	0.86	1.70	1.24	2.53
October	48.0	1.88	0.44	0.29	0.70	0.46	0.75	1.13	1.85
	38.9	34.32	9.06	5.98	13.40	8.64	14.82	19.50	21.42

(a) Estimated.

Mr. Hoyt.

TABLE 44.—(*Continued.*)

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	Loss from Land Area. Transpiration. From curve.	Loss from Land Area. Transpiration. Actual.	Loss from Land Area. Evaporation. From curve.	Loss from Land Area. Evaporation. Actual.	Total loss, in inches.	Precipitation minus total loss, in inches.	Observed run-off, in inches.
1912									
November......	33.1	0.62	....	....	0.18	0.12	0.12	0.50	0.98
December......	21.6	1.94	....	....	0.38	0.25	0.25	1.69	1.26
1913									
January........	14.0	0.59	....	....	0.35	0.23	0.23	0.36	0.94
February.......	8.4	1.02	....	....	0.18	0.12	0.12	0.90	0.84
March..........	21.6	2.61	....	....	0.68	0.45	0.45	2.16	1.18
April...........	43.8	1.90	0.42	0.28	0.82	0.54	0.82	1.08	3.70
May............	52.0	4.06	1.08	0.71	1.70	1.12	1.83	2.23	2.08
June...........	65.8	3.08	2.15	1.42	1.72	1.13	2.55	0.53	1.44
July...........	65.6	6.75	2.15	1.42	3.02	1.99	3.41	3.34	2.00
August.........	66.4	2.48	1.97	1.30	1.38	0.91	2.21	0.27	1.15
September......	56.9	3.59	1.20	0.80	1.50	0.99	1.79	1.80	1.24
October........	44.7	3.04	0.10	0.07	0.99	0.65	0.72	2.32	1.28
	41.0	31.68	9.07	6.00	12.94	8.50	14.50	17.18	18.04
1913									
November......	36.8	1.34	....	....	0.32	0.21	0.21	1.13	1.07
December......	28.3	0.14	....	....	0.21	0.14	0.14	0.00	0.88
1914									
January........	19.8	1.30	....	....	0.42	0.28	0.28	1.02	0.86
February.......	6.6	0.42	....	....	0.11	0.73	0.73	—0.31	0.73
March..........	24.5	1.65	....	....	0.62	0.41	0.41	1.24	0.75
April...........	38.5	3.05	....	....	1.05	0.69	0.69	2.36	1.99
May............	56.5	2.19	1.48	0.97	1.15	0.76	1.73	0.46	1.89
June...........	61.8	6.06	1.88	1.24	2.62	1.73	2.97	3.09	1.75
July...........	71.1	4.66	2.50	1.65	2.50	1.65	3.30	1.36	1.57
August.........	64.6	5.94	1.89	1.25	2.60	1.72	2.97	2.97	1.44
September......	58.2	2.75	1.30	0.86	1.30	0.86	1.72	1.03	1.25
October........	51.8	1.02	0.85	0.56	0.50	0.33	0.89	0.13	1.02
	43.2	30.52	9.90	6.53	13.40	9.51	16.04	14.48	15.20

Mr. Hoyt.

TABLE 45.—SUMMARY OF DATA AND COMPUTATIONS FOR WISCONSIN RIVER BASIN ABOVE MERRILL, WIS.

Drainage area, 2 630 sq. miles.

Year.	(a) Rainfall.	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	(b) Observed run-off.
1909........	25.89	7.62	5.93	13.55	12.34	14.90
1910........	22.40	6.73	5.61	12.34	10.06	9.01
1911........	35.24	9.70	6.36	16.06	19.18	....
1912........	34.32	8.84	5.98	14.82	19.50	19.52
1913........	31.68	8.50	6.00	14.50	17.18	17.55
1914........	30.52	9.51	6.53	16.04	14.48	15.22
Total......	180.05	50.90	36.41	87.31	92.74	76.20
Mean......	30.00	8.48	6.07	14.55	15.45	15.24

(a) Precipitation from November 1st of preceding year to October 31st of given year.
(b) Run-off from March 1st of given year to February 28th or 29th of following year.

Mr. Hoyt.

TABLE 46.—DATA AND COMPUTATIONS FOR WISCONSIN RIVER BASIN BETWEEN MERRILL AND RHINELANDER, WIS.

Drainage area, 1 520 sq. miles.

Year and month.	Monthly temperature, in degrees, Fahrenheit	Monthly precipitation, in inches.	LOSS FROM LAND AREA. Transpiration. From curve.	Transpiration. Actual.	Evaporation. From curve.	Evaporation. Actual.	Total loss, in inches.	Precipitation minus total loss, in inches.	Observed run-off, in inches.
1908									
November......	32.6	1.47	....	....	0.25	0.15	0.15	1.32	0.67
December......	17.4	1.09	..	....	0.20	0.12	0.12	0.97	0.56
1909									
January........	15.4	0.53	....	....	0.42	0.24	0.24	0.29	0.47
February.......	17.6	1.26	....	....	0.50	0.29	0.29	0.97	0.43
March..........	25.0	1.29	....	....	0.62	0.36	0.36	0.93	0.50
April............	34.0	2.95	....	....	0.92	0.53	0.53	2.42	2.57
May..............	51.7	2.11	1.15	0.76	1.00	0.58	1.34	0.77	4.09
June.............	65.2	3.36	2.10	1.39	1.90	1.10	2.49	0.87	1.66
July..............	66.8	5.26	2.22	1.47	2.60	1.51	2.98	2.28	0.89
August..........	67.8	2.41	2.30	1.52	1.45	0.84	2.36	0.05	0.15
September......	57.4	2.45	1.20	0.79	0.15	0.67	1.46	0.99	0.42
October.........	43.1	1.71	....	....	0.52	0.30	0.30	1.41	0.46
	....	25.89	8.97	5.93	11.53	6.69	12.62	13.27	12.87
1909									
November......	37.0	4.48	....	....	1.05	0.61	0.61	3.87	2.00
December......	16.4	0.93	....	....	0.35	0.20	0.20	0.73	0.99
1910									
January........	13.8	0.76	....	....	0.38	0.22	0.22	0.54	0.71
February.......	11.4	0.97	....	....	0.25	0.14	0.14	0.83	0.52
March..........	40.0	0.28	....	....	0.42	0.24	0.24	0.04	1.37
April............	46.8	2.46	0.72	0.48	1.10	0.64	1.12	1.34	2.16
May..............	49.2	2.42	0.92	0.61	1.15	0.67	1.28	1.14	1.34
June.............	65.7	0.38	1.15	0.76	0.45	0.26	1.02	—0.64	0.49
July..............	69.2	2.13	2.35	1.55	1.45	0.84	2.39	—0.26	0.30
August..........	66.2	3.06	1.95	1.29	1.60	0.93	2.22	0.84	0.34
September......	56.2	2.40	0.88	0.58	1.09	0.63	1.21	1.19	0.50
October...... ..	48.6	2.12	0.52	0.34	0.88	0.51	0.85	1.27	0.64
	....	22.40	8.49	5.61	10.17	5.89	11.50	10.89	11.36

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TABLE 46.—(*Continued.*)

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	LOSS FROM LAND AREA. Transpiration. From curve.	Transpiration. Actual.	Evaporation. From curve.	Evaporation. Actual.	Total loss, in inches.	Precipitation minus total loss, in inches.	Observed run-off, in inches.
1910									
November......	25.7	1.12	....	....	0.22	0.19	0.19	0.93	0.44
December......	15.9	0.83	....	....	0.32	0.19	0.19	0.64	0.51
1911									
January........	12.4	0.76	....	....	0.30	0.17	0.17	0.59	0.46
February.......	20.4	1.12	....	....	0.52	0.30	0.30	0.82	0.47
March..........	30.2	1.69	....	....	0.70	0.41	0.41	1.28	1.23
April...........	42.0	0.78	0.22	0.15	0.52	0.30	0.45	0.33	2.08
May............	59.4	4.97	1.68	1.11	2.18	1.26	2.37	2.60	1.96
June...........	69.6	1.51	2.42	1.60	1.10	0.64	2.24	— 0.73	0.88
July...........	67.8	7.05	2.30	1.52	3.30	1.91	3.43	3.62	0.25
August.........	63.8	4.72	1.80	1.19	2.11	1.22	2.41	2.31	0.39
September......	56.6	4.17	1.20	0.79	1.68	0.98	1.77	2.40	1.19
October........	43.4	6.52	....	....	1.70	0.99	0.99	5.53	4.51
	35.24	35.24	9.62	6.36	14.65	8.56	14.92	20.32	14.37
1911									
November......	24.4	2.77	....	....	0.45	0.26	0.26	2.51	1.70
December......	22.9	2.52	....	....	0.40	0.23	0.23	2.29	....
1912									
January........	—5.4	0.56	....	....	0.05	0.03	0.03	0.53	....
February.......	11.5	0.27	....	....	0.28	0.16	0.16	0.11	....
March..........	19.8	0.37	....	....	0.42	0.24	0.24	0.13	....
April...........	42.1	3.06	0.22	0.15	1.15	0.67	0.82	2.24	3.67
May............	55.2	4.89	1.40	0.92	2.00	1.16	2.08	2.81	3.56
June...........	61.6	1.78	1.88	1.24	1.08	0.63	1.87	0.09	1.29
July...........	67.1	5.44	2.22	1.47	2.62	1.52	2.99	2.45	1.76
August.........	61.8	7.84	1.62	1.07	2.95	1.71	2.78	5.06	1.90
September......	58.4	2.94	1.28	0.84	1.30	0.75	1.59	1.35	2.61
October........	48.0	1.88	0.44	0.29	0.70	0.41	0.70	1.18	1.07
	......	34.32	9.06	5.98	13.40	7.77	13.75	20.57	....

Mr. Hoyt.

TABLE 46.—(*Continued.*)

Year and month.	Monthly temperature, in degrees, Fahrenheit.	Monthly precipitation, in inches.	LOSS FROM LAND AREA.				Total loss, in inches.	Precipitation minus total loss, in inches.	Observed run-off, in inches.
			Transpiration.		Evaporation.				
			From curve.	Actual.	From curve.	Actual.			
1912									
November	33.1	0.62	....	....	0.18	0.10	0.10	0.52	0.42
December......	21.6	1.94	....	....	0.38	0.22	0.22	1.72	1.06
1913									
January........	14.0	0.59	....	....	0.35	0.20	0.20	0.39	1.06
February.......	8.4	1.02	....	....	0.18	0.10	0.10	0.92	0.80
March..........	21.6	2.61	....	....	0.68	0.39	0.39	2.22	1.16
April	43.8	1.90	0.42	0.28	0.82	0.48	0.76	1.14	4.90
May	52.0	4.06	1.08	0.71	1.70	0.99	1.70	2.36	2.31
June...........	65.8	3.08	2.15	1.42	1.72	1.00	2.42	0.66	1.56
July............	65.6	6.75	2.15	1.42	3.02	1.75	3.17	3.58	2.47
August	66.4	2.48	1.97	1.30	1.38	0.80	2.10	0.38	1.23
September	56.9	3.59	1.20	0.80	1.50	0.87	1.67	1.92	1.28
October.........	44.7	3.04	0.10	0.07	0.99	0.57	0.64	2.40	0.93
	......	31.68	9.07	6.00	12.94	7.47	13.47	18.21	18.98
1913									
November......	36.8	1.34	....	....	0.32	0.19	0.19	1.15	0.70
December	28.3	0.14	....	....	0.21	0.12	0.12	0.02	0.59
1914									
January........	19.8	1.30	....	....	0.42	0.24	0.24	1.06	0.65
February.......	6.6	0.42	....	....	0.11	0.06	0.06	0.36	0.51
March..........	24.5	1.65	....	....	0.62	0.36	0.36	1.29	0.51
April...........	38.5	3.05	....	....	1.05	0.61	0.61	2.44	2.52
May	56.5	2.19	1.48	0.97	1.15	0.67	1.64	0.55	2.35
June...........	61.8	6.06	1.88	1.24	2.62	1.52	2.76	3.30	2.20
July............	71.1	4.66	2.50	1.65	2.50	1.45	3.10	1.56	1.28
August	64.6	5.94	1.89	1.25	2.60	1.51	2.76	3.18	1.12
September......	58.2	2.75	1.30	0.86	1.30	0.75	1.61	1.14	1.09
October.........	51.8	1.02	0.85	0.56	0.50	0.46	1.02	0.00	1.19
	......	30.52	9.90	6.53	13.70	7.94	14.47	16.05	14.71

Mr. Hoyt.

TABLE 47.—SUMMARY OF DATA AND COMPUTATIONS FOR WISCONSIN RIVER BASIN BETWEEN RHINELANDER AND MERRILL, WIS.

Drainage area, 1 520 sq. miles.

Year.	(a) Rainfall.	Evaporation.	Transpiration.	Total loss.	Precipitation minus total loss.	(b) Observed run-off.
1909.......	25.89	6.69	5.93	12.62	13.27	14.96
1910.......	22.40	5.89	5.61	11.50	10.90	9.02
1911.......	35.24	8.56	6.36	14.92	20.32	....
1912.......	34.32	7.77	5.98	13.75	20.57	19.70
1913.......	31.68	7.47	6.00	13.47	18.21	18.29
1914.......	30.50	7.94	6.53	14.47	16.05	14.33
Total.....	......	......	......	......	79.40	76.30
Mean......	......	......	......	......	(c) 15.88	15.26

(a) Precipitation from November 1st of preceding year to October 31st of given year.
(b) Run-off from March 1st of given year to February 28th or 29th of following year.
(c) Not including 1911, for which there were no corresponding run-off data.

Mr. Hoyt.

TABLE 48.—RUN-OFF, IN SECOND-FEET PER SQUARE MILE, FOR A NUMBER OF WISCONSIN RIVERS.

November 20th, 1914, to March 10th, 1915.

River.	Station.	Nov., 1914.	Dec., 1914.			Jan., 1915.			Feb., 1915.			Mar., 1915.
		20-30	1-10	11-20	21-31	1-10	11-20	21-31	1-10	11-20	21-28	1-10
LAKE SUPERIOR BASIN.												
Aminicon	Aminicon Falls.	0.25	0.20	0.18	0.16	0.14	0.12	0.06	0.06	0.07	0.09	0.24
Bad	Odanah	0.45	0.48	0.26	0.18	0.18	0.21	0.16	0.16	0.23	0.28	0.29
Average, Lake Superior Basin		0.35	0.34	0.22	0.17	0.16	0.16	0.11	0.11	0.15	0.18	0.26
MISSISSIPPI RIVER BASIN.												
Namakagon	Trego	0.95	0.88	0.82	0.61	0.61	0.69	0.60	0.62	0.67	0.66	0.95
St. Croix	Swiss	0.97	0.97	0.75	0.60	0.56	0.55	0.52	0.55	0.56	0.66	0.60
St. Croix	St. Croix Falls	0.34	0.44	0.25	0.26	0.32	0.30	0.27	0.26	0.32	0.32	0.32
Chippewa	Lessard's	0.50	0.51	0.46	0.44	0.50	0.48	0.41	0.40	0.48	0.56	0.58
Chippewa	Bishop's Bridge.	0.57	0.56	0.58	0.44	0.48	0.43	0.41	0.41	0.44	0.48	0.54
Chippewa	Bruce	0.58	0.64	0.58	0.44	0.42	0.46	0.40	0.37	0.47	0.49	0.32
Chippewa	Chippewa Falls.	0.38	0.48	0.40	0.30	0.26	0.23	0.24	0.26	0.27	0.32	0.32
Flambeau	Butternut	0.70	0.75	0.68	0.60	0.65	0.64	0.63	0.58	0.58	0.59	0.59
Flambeau	Ladysmith	0.54	0.45	0.34	0.35	0.25	0.25	0.32	0.36	0.53	0.44	0.44
Eau Claire	Augusta	0.40	0.38	0.28	0.16	0.12	0.11	0.11	0.16	0.36	0.84	0.32
Trempealeau	Dodge	0.38	0.42	0.37	0.36	0.37	0.38	0.22	0.38	0.78	1.70	0.66
Black	Neillsville	0.14	0.25	0.03	0.05	0.08	0.09	0.05	0.04	0.12	0.49	0.18
Average, Mississippi River Basin		0.54	0.53	0.45	0.38	0.38	0.37	0.35	0.37	0.47	0.62	0.48
WISCONSIN RIVER BASIN.												
Wisconsin	Rhinelander	0.95	0.92	0.62	0.70	0.72	0.73	0.76	0.76	0.77	0.84	1.04
Wisconsin	Muscoda	0.49	0.50	0.45	0.40	0.38	0.40	0.38	0.41	0.72	1.15	0.84
Prairie	Merrill	0.86	0.76	0.52	0.46	0.43	0.46	0.48	0.52	0.59	0.62	.....
Little Rib	Wausau	0.12	0.18	0.10	0.08	0.06	0.09	0.10	0.12	0.45	0.52	0.10
Eau Claire	Kelly	0.30	0.42	0.22	0.20	0.20	0.19	0.19	0.22	0.25	0.34	0.29
Big Eau Pleine	Stratford	0.12	0.10	0.04	0.02	0.01	0.01	0.01	0.02	0.04	0.28	0.14
Plover	Stevens Point	1.00	1.10	0.85	0.72	0.58	0.52	0.49	0.58	0.73	0.70	0.85
Baraboo	Baraboo	0.26	0.25	0.26	0.18	0.20	0.44	0.33	0.38	0.99	2.30	0.63
Average, Wisconsin River Basin		0.51	0.54	0.39	0.35	0.31	0.35	0.33	0.38	0.57	0.84	0.54
ROCK RIVER BASIN.												
Rock	Afton	0.30	0.34	0.28	0.24	0.22	0.25	0.27	0.34	.....	.....	.....
Sugar	Brodhead	0.35	0.48	0.42	0.29	0.27	0.38	0.44	0.43	1.46	.....	.....
Pecatonica	Dill	0.35	0.34	0.33	0.32	0.30	0.28	0.26	0.50	0.95	.....	.....
Average, Rock River Basin		0.33	0.39	0.34	0.28	0.26	0.26	0.32	0.42	1.20	.....	.....
LAKE MICHIGAN BASIN.												
Menominee	Koss	0.66	0.78	0.42	0.38	0.40	0.43	0.42	0.40	0.46	0.48	0.46
Brule	Florence	0.92	0.88	0.78	0.74	0.70	0.62	0.52	0.55	0.66	0.82	0.78
Pine	Florence	0.76	0.69	0.52	0.44	0.42	0.39	0.32	0.32	0.38	0.45	0.42
Pike	Amberg	0.84	0.72	0.55	0.54	0.56	0.57	0.56	0.58	0.66	0.62	0.60
Oconto	Gillett	0.82	0.89	0.75	0.55	0.52	0.50	0.55	0.56	0.60	0.62	0.64
Wolf	New London	0.62	0.66	0.48	0.38	0.34	0.35	0.38	0.36	0.54	0.83	0.72
Average, Lake Michigan Basin		0.77	0.77	0.58	0.51	0.49	0.48	0.46	0.46	9.55	0.64	0.59

ADOLPH F. MEYER,* M. AM. SOC. C. E. (by letter).—The writer has read with considerable interest the various discussions of his paper. It was a source of some gratification to find general agreement with practically all the important principles underlying the writer's method. The only one who appears to differ in any material respect is Mr. Justin. Although practically all his contentions are either directly or incidentally met, and his criticisms answered, by Messrs. Chandler, Horton, and Hoyt, a few additional statements in reply may not be amiss. In the first place, Mr. Justin contends that the writer has not given proper recognition to the effect of slope on run-off. As pointed out by Mr. Horton, however, slope, as a factor in determining run-off, is of less importance than might at first appear. Perhaps even Mr. Justin, in discussing the paper, has over-estimated the weight given to the slope factor in his (Mr. Justin's) formula for run-off, given on page 1159. In that formula, the factor, S, representing the slope of the water-shed (derived by the illogical procedure of dividing the difference between the highest and the lowest points on the water-shed by the square root of the area, without reference to the ruggedness of the country), appears with an exponent of 0.155, which is less than the one-sixth power. On the other hand, R, representing the annual rainfall, is raised to the second power, and T, representing the mean annual temperature, appears in the denominator with the first power. In other words, Mr. Justin considers rainfall about thirteen times as important a factor as slope. It is very evident, then, that even Mr. Justin does not, in his formula, give to the slope factor the importance which one would infer is attached to it from his criticism of the paper. Mr. Meyer.

The writer takes exception to Mr. Justin's interpretation of his comments relating to the necessity for accurate estimates of the monthly distribution of run-off. Accuracy, in this connection, as in most others, is a relative matter. The writer does not subscribe to Mr. Justin's interpretation of desired accuracy in monthly distribution of run-off, merely because even a rather approximate monthly distribution does not lead to gross errors in determining storage from a mass-curve constructed on such a small scale as to preclude the possibility of accurate reading. Moreover, he does not agree that Mr. Justin has shown, by his use of the mass-curve, that monthly temperatures need not be taken into account in estimating monthly run-off, in fact, he desires to reiterate most emphatically the opposite view.

Mr. Justin next credits the writer with over-emphasizing the importance of the long-term mean as against the long-term record, on pages 1138 and 1139. Referring to these pages one reads:

* Minneapolis, Minn.

Mr. Meyer.

"On comparatively few streams of the country do the records of discharge extend over a long term of years. Short-term records do not give the extremes of high and low flow unless by sheer accident such years have been included in the term over which observations extend. Short-term records, moreover, do not give a satisfactory value for mean utilizable flow. In the last analysis, it is usually necessary to supplement the observed stream-flow data with computed values based on rainfall and other physical data, in order to arrive at a probable maximum, minimum, and mean utilizable flow for any given stream.

"In order to show the annual and periodic variations in rainfall and run-off on the two streams, considered in this paper, for which relatively long-term records are available, the curves in Figs. 34 to 36 have been prepared."

This quotation by itself appears to reply to Mr. Justin's criticism.

Mr. Justin next refers to the writer's subdivision of losses out of rainfall into evaporation losses from land and water surfaces and transpiration losses, and comments on this subdivision as being "both an unfortunate and unnecessary complication of the subject". Here, again, the writer's position is fully approved by the discussions of Messrs. Chandler, Horton, and Hoyt. Mr. Justin has either failed to read or has misconstrued, the writer's discussion of the subject of transpiration, particularly in that he concludes that all the experiments of the United States Department of Agriculture fail to differentiate between transpiration and evaporation. In reference to this matter, Mr. Justin is respectfully referred, in particular, to *Bulletin 285*, Bureau of Plant Industry, on "The Water Requirements of Plants", which gives a full review of the literature on the subject.

Mr. Justin next refers to the possibility of deriving a formula for the transpiration of any given water-shed by representing the following factors mathematically,

"Thriftiness of inhabitants (as a coefficient to be determined by judgment); tons of hay, bushels of corn, oats, rye, wheat, etc., density of population per square mile, percentage of farmers who are graduates of agricultural colleges, tons of fertilizer per acre, gallons of alcohol consumed per capita per year, etc., etc."

Such views require no comment.

Mr. Justin then refers to the transpiration curve, commenting,

"Although the author does not particularly emphasize the fact, it is clear that this curve is not based on observed data, but on the assumption, that 'the law first stated by Van't Hoff and Arrhenius, that most chemical reactions and physiological processes double in activity for every rise in temperature of 10° cent.' is also applicable to transpiration."

If Mr. Justin would take time to investigate, he could readily determine that the law of Van't Hoff and Arrhenius is not a mere

assumption. If he will even refer to the bottom of page 1089 of the paper, he will find at least one reference to the verification of this law in connection with transpiration phenomena. Mr. Meyer.

Those who have given the paper an unbiased reading, or who have had some previous knowledge of the subject of transpiration, will hardly agree with Mr. Justin's sweeping conclusion, stated in these terms:

"Thus, it is demonstrated that the differentiation between evaporation from ground surfaces and transpiration is a matter which is still in the realm of surmise."

Perhaps the best measure of the manner in which Mr. Justin has apparently perused the author's paper is afforded by the following quotation:

"When the author compiled Table 5, he had before him the recorded rainfall and run-off for the various water-sheds considered. He then manipulated his curves, his coefficient, and his judgment to derive the three quantities, evaporation from water surfaces, evaporation from ground surfaces, and transpiration, always bearing in mind the fact, subconsciously or otherwise, that the sum of the three must approximately equal the difference between run-off and rainfall. Nothing could be more simple."

Such insinuations of the juggling of figures, as those made by Mr. Justin in this paragraph, are no more worthy of comment than his remarks respecting the consideration of such matters as educational training and alcohol consumption in connection with the determination of transpiration from a given water-shed, for the purposes of estimating stream flow.

If Mr. Justin still believes that the intricate relationship between such natural phenomena as rainfall and run-off can be expressed by a simple mathematical formula, he is welcome to that belief. In his paper on "Derivation of Run-Off from Rainfall Data",* however, he did not prove that such a simple relationship existed, even though his formula gave reasonably accurate results on the limited number of water-sheds, having a relatively constant annual rainfall of from 35 to 50 in., to which it was applied. The writer has applied his method, with good results, over a considerably greater range of natural phenomena. A formula of the kind used by Mr. Justin can never give more than approximate results, and then only on water-sheds where the rainfall is considerably more than sufficient to supply all ordinary evaporation and transpiration losses. The writer is supported in this view by several of the other engineers who have discussed the paper, and by the opinions of hydraulic engineers as recently expressed in other technical literature.

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 346.

Mr. Meyer. Mr. Justin will find, on page 1118, at the foot of Table 7, in which the column heading "Actual Evaporation" is first used, a note stating exactly what this column headed "Actual Evaporation" refers to, without finding it necessary to warn those who actually have read, or will read, the paper that this column represents "merely the author's estimate", and not actual measured evaporation.

Mr. Strong's discussion is interesting and to the point, and requires no special comment. The emphasis laid on judgment, experience, and a thorough knowledge of the laws governing evaporation, transpiration, and underground flow, combined with a knowledge of the physical data for the drainage area under consideration, in order to be able to make rational estimates of run-off, is timely, and entirely in accord with the writer's view.

Mr. Chandler evidently agrees with the writer's chain of reasoning, and with the fundamental principles involved in his method of computing run-off. It is to be regretted that Mr. Chandler did not have time to complete the tabulations involved in an application which he appears to have made, of the writer's method, to several Dakota streams.

The writer is entirely in accord with the second point to which Mr. Chandler calls attention. Better results will be secured in every case, particularly on those water-sheds where the rainfall is less than 25 in., on southern water-sheds where the losses are high, and on all water-sheds having considerable portions which differ widely in physical characteristics, by subdividing the drainage basin above the point at which estimates of run-off are desired. If rainfall and run-off bore a direct relation to each other, there would be no harm in averaging the monthly rainfall records for a large water-shed. It is contrary to the fundamental principles underlying the writer's method, however, to average greatly varying quantities of precipitation for the purpose of deducing an average for a large water-shed. In fact, he desires to confess that he made the rough application of his method to the large water-shed of the Colorado River at Austin, Tex., more from curiosity than with the expectation of deriving rational results. While making the computations for this water-shed, and noting where the computed precipitation minus losses were negative and where, nevertheless, considerable surface run-off appeared in the stream, he frequently had occasion to refer to the precipitation records and to note that in such instances there invariably was excessive precipitation on one or more minor tributaries of the stream, and little or no precipitation over the drainage basins of other tributaries.

The writer also agrees with the third point made by Mr. Chandler, on page 1163. He desires to add, however, that even the discharge

records of the United States Geological Survey (at least for Minnesota streams) do not take into consideration lake storage, but give merely the run-off from the water-shed above the station, even though such run-off represents little else than outflow from a lake. Failure to take lake storage into consideration has led to grossly inaccurate conclusions, with respect to the possibility of reservoir control of stream flow, in several mass-curve studies with which the writer is familiar. Mr. Meyer.

The only important point of criticism made by Mr. Chandler with which the writer does not entirely agree, is the statement on page 1164 to the effect that as small an error as 5% in the assumed evaporation coefficient might double or treble the computed run-off, or might wipe it out entirely, in such regions as the Northwestern Central States. Such a result as that feared by Mr. Chandler would be secured only in case the transpiration losses were not properly reduced during months of deficient rainfall and deficient storage of water in the upper few feet of soil, or on those water-sheds from which the normal run-off is only about 1 in. or less. The writer has frequently expressed his firm belief in the necessity for meter measurements of the discharge of streams, particularly in the case of streams in which the annual discharge, measured in inches of depth on the tributary water-shed, is exceedingly small.

Mr. Grunsky points out the surprising uniformity in the quantity of run-off from a water-shed on which the rainfall approximates or exceeds 40 in. per annum. He presents a curve (Fig. 37) based on a relatively simple and direct relationship between rainfall and run-off. Although this curve may give satisfactory results for the California conditions under which it was derived, its application must of necessity be limited, because it fails to recognize, in an adequate manner, the various factors which determine the run-off from any given water-shed.

Mr. Horton has given an interesting and instructive summary of factors modifying stream flow, and of different methods at present in use for computing run-off from a given drainage basin.

It is gratifying to find an engineer of Mr. Horton's standing coming forward with a clear-cut statement in favor of a method of estimating run-off which takes into consideration, not only rainfall, but other physical data pertaining to the drainage basin, such as temperature and the physical characteristics and cultural conditions of the water-shed.

Perhaps the writer failed to state with sufficient clearness, or perhaps Mr. Horton misinterpreted the reference to the results of the experiments of Transeau, given on page 1085. Mr. Horton points out that these experiments appear to indicate greater evaporation from

Mr. Meyer. a bare gravel slide than from any other condition of the soil. The writer's statement, introducing these experimental data, is as follows:

"It is a well-known fact that all forms of vegetation, particularly forests, shade the ground to a certain extent, and consequently reduce the rate of evaporation of free moisture. Whether or not they reduce the total quantity evaporated per month, or per year, depends also on the relative rates at which the rainfall can percolate into the forest floor, the cultivated field, and the bare ground, or run off into the streams."

It appears to the writer that this statement anticipates Mr. Horton's remark to the effect that:

"Actually, the evaporation, particularly for the gravel slide, is of short duration, and in some cases is continuous for the other soil conditions cited. The rate may be greater, but the total evaporation is less for the gravel slide than for the other conditions."

This statement is entirely in accord with the writer's views just quoted, and repeated in other portions of the paper, as for example, on pages 1081 and 1086.

It is to be regretted that Mr. Horton did not present more of the details of his method of estimating run-off. It is noted that he further subdivides rainfall losses by using the factor "interception" loss. In the writer's method, interception is taken account of in the evaporation coefficient, as indicated on pages 1085 and 1086. In the case of water-sheds covered with a dense growth of coniferous trees, the writer also uses a larger evaporation coefficient during the winter, when the precipitation occurs as snow, than during the summer, and larger, also, than would be used in case the water-shed were covered with deciduous trees.

Mr. Horton's comments as to the lack of effect of slope on the interception loss is in accord with the writer's views. "Evaporation opportunity" appeals to the writer as an apt expression.

Mr. Horton expresses the opinion that the writer's method does not adequately cover or account for transpiration losses. This may be entirely true, yet the basis for improvement does not appear to be afforded by Mr. Horton's discussion. "Crop yield" is referred to as the best and simplest measure of transpiration loss generally available. This may be substantially true, when considering the average transpiration over a period of years from cultivated water-sheds, and to this extent is utilized by the writer in determining the normal seasonal transpiration (pages 1093 and 1102). Crop yields, of course, are of little assistance in estimating transpiration from timbered or brush-covered water-sheds. Crop yields, in the case of grain, also fail to take into account differences in the yield of straw, which may result in

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very substantial differences in the transpiration loss. The relation between transpiration loss and vegetable matter produced is frequently referred to in the paper; nevertheless, monthly rainfall and temperature, and quantity of moisture stored in the soil, are believed to afford a better practical index to monthly transpiration losses than crop yields, on most water-sheds in the United States.

The writer was very much pleased to find that at least one engineer felt that it was necessary to make an actual application of the writer's method of computing run-off in order to be able to pass fair judgment, and has presented the detailed computations involving this application as a portion of his discussion. Although the results of Mr. Hoyt's computations bear minor modification, they nevertheless indicate that reasonably satisfactory estimates of run-off may be made by this method, by any engineer who takes the necessary time to familiarize himself with it even as briefly presented in the paper.

Mr. Hoyt states that, in the introduction to recent papers on the surface water supply of the United States Geological Survey, a note has been added, calling attention to the fact that the figures showing depth of run-off, in inches on the water-shed, may be subject to gross errors resulting from the inclusion of large non-contributing districts, or from the omission of estimates of water diverted for irrigation, or other purposes, from a portion of the water-shed, and that consequently, run-off values have not been stated in inches of depth on the drainage area for all streams having water-sheds on which the annual rainfall is less than 20 in., nor on drainage areas for which such figures of run-off might be misleading on account of stream flow diversion, etc. The writer agrees entirely with the Geological Survey in the desirability of adding this warning. It is well to have in mind, however, that the daily discharge records, as published by the Geological Survey, of the streams to which this warning applies, merely constitute a report of what has happened in the past, and offer no adequate indication of what may occur in the future at the given point on the stream, or on its tributaries. If the work of stream gauging is to be confined to an historical study, then such published data of stream discharge are valuable as history, but if the results of stream gauging are to be used as the basis for the prediction of the quantities of water which will probably be available from month to month during succeeding years, at different points on the basin of which the run-off was gauged, then some basis for making extensions and analyses of records must be found. For example, the basic principle underlying the control which is being exercised over the flow of the stream, by the given reservoirs, must be determined. The extent and location of present diversions from the water-shed must be known with at least reasonable accuracy, and estimates of future diversions must be made. The uses to which the diverted water is

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put must be known, so that estimates may be made of the probable future diversion from month to month under varying quantities of monthly precipitation. If large portions of a drainage area are non-contributing under certain meteorological conditions, this fact must be determined, and the reason for changes in the quantity of water contributed by certain portions of the water-shed must be at least reasonably well determined. The writer contends that, even on such water-sheds as those referred to in the note quoted by Mr. Hoyt from the recent Water Supply Papers, estimates must be made of probable future yields of water. Such estimates can only be made on the basis of past records of stream flow, properly analyzed in connection with all the available physical data for the given water-shed.

In the writer's estimation, the records of daily discharge published by the Geological Survey, for such streams as those just referred to and which, for the reasons given, are not reduced to inches of depth on the water-shed, are of little value as the basis for expenditures for works of improvement which are in any way dependent on stream flow, until these records of discharge have been given detailed study and analysis in connection with physical data for the given water-shed. Certainly, past records of stream flow are of no help in determining the probable future yield of water from the given drainage basin until the hydrological phenomenon and physical conditions which have resulted in the given observed yields, have become known.

The very essence of the writer's method of computing run-off from rainfall and other physical data is a careful study of the physical characteristics of the given drainage basin. Where the portion of the drainage basin of any given stream, which does not ordinarily contribute to the run-off, forms a large percentage of the entire drainage basin above the point at which estimates of run-off are desired, such a drainage basin should be subdivided, and estimates should be made of the probable run-off from the several "homogeneous" portions of the basin, by a detailed study of each, instead of by averaging precipitation, temperature, cultural, and other physical conditions over the entire water-shed, and applying a single coefficient. In reference to this, however, it should be noted that in the paper a study was made of three California streams having drainage basins which differed widely. The method was applied to two tributaries, and then to the parent stream, which included the tributaries for which run-off was separately estimated, and though the coefficients of evaporation for the two tributaries were 0.60 and 1.10, respectively, and the coefficient used for the parent stream was 0.85, the computed annual precipitation minus loss for both the tributaries and the parent stream compares surprisingly well with the observed run-off, indicating, though it does not prove, that even where it is necessary to average widely different physical characteristics on a given water-shed, and

to use a single coefficient for a relatively large water-shed, it may, nevertheless, be possible to secure surprisingly close estimates of run-off, from rainfall and other physical data, by the writer's method. **Mr. Meyer.**

The writer desires to emphasize the point made by Mr. Hoyt, that though a run-off year beginning on March 1st corresponds very well with a rainfall year beginning on November 1st, on a number of the streams of Minnesota studied by the writer, these water years are not applicable to streams on which any substantial quantity of surface run-off occurs during the winter. On such streams, a water year beginning about September 1st is more satisfactory.

Mr. Hoyt desires to know why the writer did not use the evaporation observations made at Madison and Menasha, Wis., and at Iowa City, Iowa. The only indication of the reason for the omission of the records to which Mr. Hoyt refers is given on page 1077 in the following words:

"Many other records of evaporation are available, but are omitted here because they do not give the evaporation from shallow water under conditions of wind and humidity prevailing throughout the Northwest."

The observations of evaporation made at Menasha, Madison, and Iowa City give the evaporation from relatively large, deep bodies of water. If Mr. Hoyt will plat these records on the curve of evaporation from water, snow, and ice, Fig. 12, which gives the evaporation from open bodies of water of medium size and depth, he will find that the records at the stations referred to agree more nearly with this curve than with the curve of evaporation from shallow water. This curve of evaporation from shallow water, Fig. 8, was used in connection with the construction of the curves of evaporation from land areas. In view of the fact that the temperature of all relatively large or deep bodies of water is considerably lower than that of the air, as observed by the Weather Bureau, during the spring, and higher during the fall, it would clearly have been wrong to have used any records of evaporation from other than shallow bodies of water, in connection with the determination of the evaporation of water from land areas, the temperature of which more nearly approximates the air temperature as observed by the Weather Bureau.

Mr. Hoyt requests more detailed information in regard to the development of the curve of evaporation from land areas. Most of the basic facts and considerations are presented in the paper, and there is little more to be said, except that the curve represents the result of a number of revisions following trial applications of the method to typical water-sheds. The curves have gone through four or five revisions, and those presented were adopted because they gave the best results when actually applied in estimating stream flow.

It is very true, as Mr. Hoyt states, that the run-off resulting from a given quantity of rain falling in short intervals, differs radi-

Mr. Meyer. cally from that resulting from well-distributed rains, yet it is also true that the normal distribution of rainfall, when the month is used as a basis, is reasonably similar, year after year, in any given locality and for the same month of the year. These ordinary conditions are taken account of in arriving at the proper evaporation coefficient. Extraordinary or excessive rainfalls must be given separate consideration. The writer's method, as presented, furnishes, primarily, a skeleton of basic principles and steps in the computation of run-off. There is practically no limit to the degree of refinement to which the computations may be carried by taking into account, in the computation of losses for each month, variations from the normal meteorological phenomena for the given water-shed. Whenever warranted by the importance of the results to be secured, daily records of temperature and precipitation should be used to assist in computing monthly evaporation and transpiration losses.

Mr. Hoyt regrets that the writer has not given the transpiration and evaporation coefficients used for the several drainage basins to which the method of computing run-off has been applied. The "evaporation coefficients" are given in Table 5. No transpiration coefficients are given because the transpiration, as taken from the transpiration curve, must be further corrected, as indicated in the outline of the method on pages 1101 to 1103, under "II-C-3", for deficient moisture supply.

In Mr. Hoyt's application of the writer's method, no corrections have apparently been made for deficient rainfall and soil storage in arriving at the monthly transpiration. For example, on the Rock River water-shed, in September, 1908, Mr. Hoyt used the full transpiration loss of 1.59 in., which is almost exactly equal to the precipitation for that month, and which resulted in a negative "precipitation minus loss" for that month of more than 1 in. September followed 3 months of deficient precipitation, the losses for July being credited with more than 1 in. of moisture derived from soil storage. Now, it stands to reason that if the rainfall for every month, since May, has been insufficient to supply the normal requirements of evaporation and transpiration, there will not be sufficient moisture available in the soil to permit full normal transpiration during September, even if plant growth had not been stunted as the result of insufficient rainfall during the entire season. The transpiration for that season should have been reduced at least from 1 to 1.25 in. The computed annual precipitation minus losses for this water-shed during 1907 should also have been reduced by about 1 to 1.25 in., and that for 1908 should have been increased by a similar quantity on account of evident changes in ground storage.

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The writer is unable to furnish any further information with respect to the coefficients used on the several water-sheds treated in the paper. The coefficient adopted for each water-shed was used throughout the entire period of years over which run-off was computed, consequently there is no such variation from year to year as one would infer from Mr. Hoyt's comments. If the coefficient had been varied from year to year, it would, of course, have been possible to have made the computed run-off agree exactly with the observed. All inaccuracies in the method are thrown into the final result, which is the computed annual run-off, and has been placed beside the observed annual run-off, so that the equivalent of the information which Mr. Hoyt requests is already given in the paper.

Mr. Hoyt gives a table of coefficients (Table 36), which he selected because they gave the most consistent results. The writer has never attempted to go to the refinement of expressing coefficients to single hundredths, such as 0.59, used by Mr. Hoyt. In this case the writer would use 0.60, and instead of 0.66 he would use 0.65, because the use of coefficients stated to single hundredths is not warranted even by the run-off data from which Mr. Hoyt derived his values. As a general criticism, the writer would state that Mr. Hoyt used slightly larger transpiration losses than would be indicated by the values given in the paper for approximately similar cultural and light conditions.

The writer believes that Mr. Hoyt's Table 37, indicating the coefficient which it would have been necessary to apply to the combined values as taken from the evaporation and transpiration curves in order to secure the necessary loss so as to leave a precipitation minus loss equal to the observed run-off, offers no information which affords any criterion by which to judge the accuracy of the writer's method. The variation in coefficients indicated is particularly misleading, as all changes due to soil and subsoil storage are thrown into the coefficient. The proper basis for comparison is computed with observed run-off, using the same coefficient throughout the entire period of years, unless radical changes in cultural conditions occurred on the water-shed.

Mr. Hoyt apparently has misinterpreted, or perhaps the writer has failed to state with sufficient clearness, the ordinary range of evaporation coefficients for the Northwest. In reference to the evaporation coefficient, it is stated (page 1100), "This coefficient ranges from about 0.95 to 1.25 for most water-sheds of the Northwest, and for similar ones elsewhere". Then the factors on which the coefficient depends are summarized, and then there is the statement: "Between these extremes the usual working values will be found". Evidently 0.95 to 1.25 represent usual working values for this coefficient "for most water-sheds of the Northwest, and for similar ones elsewhere".

Mr. Meyer. The three Wisconsin River water-sheds considered by Mr. Hoyt are so sandy, according to the descriptions quoted from Mr. Stewart, as to permit no surface run-off during the open season; hence they are clearly of an exceptional character. The values, 0.95 to 1.25, as stated before, represent ordinary working values for most water-sheds of the Northwest. They do not represent the coefficients which would be used for either a sand pit or a city pavement. It is entirely unnecessary for the writer to find "causative errors" in Mr. Hoyt's computations, neither is it "unfortunate" that the sandy water-sheds of the northern Wisconsin River Basin require the use of a small evaporation coefficient.

If the writer gave the impression, as one might gather from Mr. Hoyt's statements on page 1179, that his method of computing monthly run-off was "largely a matter of judgment in adding to and subtracting from the 'precipitation minus losses', as determined from the various curves", it was unfortunate, because quite contrary to the facts.

The paper presents three curves which form the basis for estimating the monthly distribution of run-off for the Root River watershed, and gives the detailed computations for this water-shed so that the method may be clear. It would appear, from the computations presented for this water-shed in Table 35, that, for a considerable portion of the year, the stream flow consists entirely of seepage. The determination of seepage flow, however, involves no estimate whatever, the values being taken directly from the curve of Fig. 32. Even if these three sets of curves were rigidly applied, the result would still be a rational monthly distribution of run-off, although the quantities secured in that way would not agree as well with the observed run-off as the writer's computations, which take daily temperature, precipitation, etc., into consideration, as indicated by the column, in Table 35, headed "Notes".

It is impossible to give detailed information regarding the development of these curves of Figs. 30, 31, and 32, because they are not based on any definite group of data, but on a process of logical reasoning, experience, observation, and all the facts bearing on the subject which the writer could command. These curves would necessarily have to be changed before being applicable to different streams, although the change would be small for some water-sheds. On a steeper, more impervious water-shed, for example, the percolation curve in Fig. 30, which now has a value of 2 in., might be given a value of 0, and the curves having values of 3 and 4 in., respectively, might receive values of 1 and 2 in., although these curves would be more closely spaced. In a similar manner, the curves of Fig. 31 would receive different values. For a perfectly impervious, sloping watershed, the curve of zero soil storage would be approximately in the position of the curve of 4 in. soil storage in this figure.

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The frequency curves presented by Mr. Hoyt are not frequency curves of computed monthly run-off, as one would infer from his statements. They merely represent monthly precipitation minus losses, and, as such, are interesting in that they show the equalizing effect of soil and surface storage on stream flow. Even though not intended for this purpose, these curves are perhaps the best graphical proof of the lack of a simple, direct percentage relationship between precipitation and run-off that could be presented, and all believers in exponential formulas for determining run-off from rainfall are respectfully referred thereto.

Frequency curves of computed monthly run-off and observed monthly run-off, for the Root River, for example, are almost identical when covering the same period of time, although the frequency curve of computed monthly run-off, based on 20 years of rainfall and other data, is radically different from that based on the short-term observed run-off records. This, again, emphasizes the need for supplementing and extending short-term observed stream-flow records.

The writer does not hold that it is possible to make satisfactory determinations of the monthly flow of all rivers. He does, however, believe it to be possible to make fairly accurate determinations of the monthly run-off from by far the greater portion of the water-sheds of the country, on the basis of reasonably complete physical data.

Table 48, showing run-off from a number of Wisconsin water-sheds during the winter, summarizes valuable data. The writer also holds in high regard Mr. Hoyt's discussion of the effects of ice on stream flow, as given in Water Supply Paper 337. Bi-monthly meter gaugings taken in connection with accurate temperature records, undoubtedly furnish a better basis for the estimation of winter discharge of streams than computations based on physical data alone. This fact does not prove, however, that there is no need for making estimates of winter stream flow on the basis of physical data where the available discharge records extend over only a few years. In fact, the necessity for such estimates would seem to be increased rather than diminished by Mr. Hoyt's own arguments. If the winter estimates of stream flow made previous to the publication of Water Supply Paper 337 are less accurate than those made since then, the need for extending and supplementing the records now being obtained would appear to be increased rather than diminished. It is so easy to lose sight of the fact that records of the past discharge of streams are primarily for the purpose of predicting what the probable yield of the same water-sheds will be in the future.

The writer has repeatedly emphasized the fact that his method of computing run-off is to be used primarily for the purpose of analyzing, supplementing, and extending observed stream-flow data. It is practically always possible for an engineer to obtain from a few months'

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to a few years' gaugings on a given stream before it is necessary to make any report whatever on the probable flow. On the basis of the available records of discharge and an intensive study of the given water-shed, reasonably accurate values for the evaporation and transpiration coefficients can be determined, and in this way an estimate can be made of the probable yield of the given water-shed, by the rainfall-loss method, which is far more reliable, in the writer's estimation, than a conclusion based merely on short-term records of discharge.

Mr. Hoyt states that the extreme maximum and minimum run-off cannot be determined at all accurately by the writer's method. That may be very true when maximum and minimum values of discharge, in cubic feet per second, are viewed in the light of 25 or 50 years of accurate run-off records. On most streams, however, the available stream-flow data cover such a short period of years that the writer believes a more accurate estimate of flood flow can be made on the basis of physical data properly analyzed in connection with the available records of stream flow than the data afforded by the short-term records alone.

The writer does not believe, for example, that Mr. Hoyt would contend that the maximum flood flow on the Black River at Neillsville can be more accurately determined from the 4½ years of run-off records than from even the 9 years of computed precipitation minus loss, given in the paper for this stream. No run-off records are available for 1911, yet, according to Mr. Hoyt's figures, the computed precipitation minus loss for September was 3.67 in., and for October 7.27 in. The next highest computed precipitation minus loss for the summer or fall given in Mr. Hoyt's paper, is 4.14 in. for June, 1914, during which a total run-off of 4.11 in. was observed.

Though the writer has not had time to make a careful estimate of the probable maximum rate of run-off during October, 1911, it would appear from the large computed precipitation minus loss for September and October that the rate must have been very substantially larger than that which occurred in June, 1914. The daily precipitation records indicate that there was a precipitation on the Black River water-shed, above Neillsville, of about 1.8 in. on September 27th, and of about 0.5, 1.5, and 3.0 in. on October 1st, 3d, and 5th, respectively. Evidently, conditions were exceptionally favorable for an extreme flood. The rainfall between September 27th and October 3d, more than exhausted all the available soil storage capacity, so that most of the 3 in. of rain which fell on October 5th inevitably found its way into the stream, with the result that both The Dells and the Hatfield Dams failed, and a large portion of the business district of Black River Falls was swept away.

Although, as stated on page 1138, "The determination of the probable extremes of discharge for streams on which only short-term run-off

records are available is too large a problem to be discussed in the present paper", it may bear reiteration that such estimates must be made on the basis of the available records of stream flow, properly analyzed and supplemented by estimates based on rainfall and other physical data. It may here be remarked that, in making such estimates of probable flood flow, the writer has found it desirable to utilize isohyetal charts of excessive precipitation for the region in which the stream, for which the estimates of flood flow are desired, is situated. Such charts can be compared with the rainfall which produced the highest discharge during the period over which run-off records extend, as an aid in estimating the probable extreme flood. Mr. Meyer.

In discussing Mr. Hoyt's computations for the Black River water-shed, it may be well to call attention to the fact that the total computed precipitation minus loss from November, 1905, to April, 1906, inclusive, is only 5.00 in., whereas the observed run-off for the same period is given as 7.46 in. It requires no argument to show that this represents an impossible condition—either rainfall or run-off records being grossly incorrect.

No comparison can be made between annual computed precipitation minus loss and observed run-off for the Wisconsin River water-sheds, because about 45% of the drainage area above Rhinelander and about 25% of that above Merrill is under reservoir control. It will be noted, however, that the computed values of precipitation minus loss for the entire period agree very well with the observed run-off.

The annual computed precipitation minus loss for the Wisconsin River water-shed, between Rhinelander and Merrill, only about 5% of which is under reservoir control, agrees very well with the observed annual run-off, except during 1911. It is difficult to see, however, even while recognizing the sandy character of the Upper Wisconsin River water-shed, why the precipitation which occurred during July and August, 1911, did not produce more run-off than that given in Table 46. For example, the run-off decreased from 0.88 in. in June to 0.25 in. in July, and 0.39 in. in August, even though the precipitation increased from 1.51 in. in June to 7.05 in. in July, and 4.72 in. in August. If these figures are correct, and merely reflect the absorptive capacity of the Upper Wisconsin River water-sheds, surely no further argument is needed to prove that these water-sheds are indeed exceptional, and that the evaporation loss is necessarily very small.

The writer has not taken time to check Mr. Hoyt's figures, but it appears that in some cases the estimates of average precipitation for the different water-sheds will bear some revision. For preliminary computations, it may be well enough to average, arithmetically, and without weighting, the recorded precipitation for all stations on or near the given water-shed. Using the average of all available precipitation records on a given water-shed may also be satisfactory where the

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stations are numerous and well distributed. Where stations on a water-shed are few, however, and records of stations adjacent to the water-shed are used, they should not be given the same weight as those for stations situated centrally on the water-shed.

The writer intended to state in the paper that the rainfall and run-off records for the Ohio, the James, and the Roanoke Rivers were taken from Mr. J. C. Hoyt's paper, "Comparison Between Rainfall and Run-off in the Northeastern United States".*

Since the writer's paper was published, revised monthly run-off values for the Root River, at Houston, from March, 1913, to February, 1914, have been furnished by the Washington Office of the United States Geological Survey. These figures are as follows:

March	1.20 in.
April	0.46 "
May	0.39 "
June	0.30 "
July	0.43 "
August	0.30 "
September	0.22 "
October	0.25 "
November	0.22 "
December	0.23 "
January	0.24 "
February	0.17 "
Total	4.41 in.

Perhaps the most striking fact about these revised figures is that the run-off for March, 1913, as given by the Washington Office, is less than half that given by the District Office, as a preliminary figure. The difference in estimates no doubt arises mainly from the fact that gauge heights for the month are confined to the period from March 22d to 31st. The writer is inclined to accept his estimates, based on rainfall and other data, as being perhaps more accurate than either the estimate of the District Office or that of the Washington Office of the Geological Survey.

In conclusion, the writer desires to express his appreciation of the interest shown by those who have discussed the paper, and gratification at the general agreement with the fundamental principles underlying his method. He desires, further, to reiterate the often expressed sentiment that all methods of computing run-off from rainfall and other physical data should be used, primarily, for the purpose of analyzing, supplementing, and extending observed stream-flow records, so as to make these records a better basis for works of improvement into which run-off enters as a factor.

* *Transactions*, Am. Soc. C. E., Vol. LIX, p. 431.

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TRANSACTIONS

Paper No. 1349

TEMPERATURE CHANGES IN MASS CONCRETE*

BY CHARLES H. PAUL, M. AM. SOC. C. E., AND A. B. MAYHEW, ASSOC. M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. GEORGE T. SEABURY, WILSON FITCH SMITH, G. IMMEDIATO, GARDNER S. WILLIAMS, WALTER M. SMITH, A. J. WILEY, J. WALDO SMITH, AND CHARLES H. PAUL AND A. B. MAYHEW.

SYNOPSIS.

If free to move, all structural materials undergo changes in length due to changes in temperature, and, if not free to move, a stress in the material is produced equal to that required to produce the deformation in length. In order to determine the extent of this change in length, or the corresponding stress in the material, it becomes necessary to know the range in temperature to which the material is to be subjected. In large masses of masonry, the changes in temperature at varying distances from the exposed faces are not known, and, largely due to this fact, the changes in length of large masses of masonry, together with the actual internal distribution of temperature stresses, are among the most indefinite factors in the design of such structures.

The purpose of this paper is to place before the Society the results, thus far obtained, of experiments being made to determine the changes of temperature in the concrete of the Arrowrock Dam, together with a description of the apparatus used.

Although some of these experiments have been in progress for more than a year, the greater part of the results is still affected by the high

* Presented at the meeting of May 5th, 1915.

temperatures produced by chemical action while the cement is setting, and, therefore, this paper will be largely descriptive, and should be considered in the light of a preliminary paper.

Among the conclusions already reached are the following:

(1) Large bodies of concrete deposited rapidly during a summer season develop a temperature of from 90 to 95° within a period of about 30 days, and maintain nearly that temperature for several months.

(2) In the case of concrete 1 ft. from an exposed face, there is a daily variation in temperature of about 2° when the daily variation in the temperature of the air is about 50 degrees.

(3) In the case of concrete 2 ft. from an exposed face, there is a daily variation of less than 1° when the daily variation in the temperature of the air is about 50 degrees.

(4) In the case of concrete 3.5 ft. from an exposed face, no daily variation in temperature is apparent when the daily variation in the temperature of the air is about 50 degrees.

(5) The seasonal variation in the temperature of concrete 3.5 ft. from an exposed face is about 32° when the seasonal variation in the mean daily temperature of the air is about 75 degrees.

(6) The experiments have not yet been carried far enough to show the seasonal variation at other distances from exposed faces, but it is probable that they become very much less as the distance from the face increases.

INTRODUCTORY.

The Arrowrock Dam, being constructed by the United States Reclamation Service near Boise, Idaho, and on which the writers are employed as Construction Engineer and Principal Engineering Assistant, respectively, is of the gravity type, and is curved in plan to a radius of about 662 ft. at the center of the crest. Inasmuch as little information is available concerning temperature changes in large bodies of masonry, and as this is often an important factor in the design of high dams, it was decided, for the benefit of future designs and to further engineering knowledge in general, to install in the Arrowrock Dam devices for recording the temperature of the masonry at various distances from the exposed faces. Nine of these devices

have been installed to date, and have been in place for periods ranging from about 3 to about 19 months.

It will probably be several years before a complete study can be made of the temperature changes, but, as the data on this subject are so meager, it is deemed advisable at this time to place before the Society the results thus far obtained, together with a description of the apparatus used and the principal conditions affecting the temperature readings.

TEMPERATURE RECORDING DEVICE.

The electrical resistance thermometer was considered to be the simplest and best apparatus for the purpose contemplated. The fundamental principle of the resistance thermometer is the fact that a change in the temperature of a metal causes a corresponding change in its electrical resistance.

The thermometer, or resistance bulb, consists of a coil of fine copper wire, encased in a metal tube about $\frac{3}{8}$ in. in diameter and about 5 in. long. One end of the tube is closed and the other end is soldered, with a water-proof joint, to a lead-covered cable containing the wires connecting the bulb with the switch-board to which the temperature indicator is attached.

The leads connecting the thermometer to the switch-board are constructed as follows: Each wire of the cable is insulated with three wrappings of silk, thoroughly saturated with beeswax after wrapping. The three strands of which the cable is formed are then brought together and encased in one wrapping of cotton and one of braid similarly water-proofed. The cable is then drawn into the lead armor, the latter being soldered directly to the brass tube containing the resistance coils.

The temperature indicator consists of a modified Wheatstone bridge calibrated to read temperature units instead of ohms. The dial is direct reading in degrees, Fahrenheit, with a range from zero to 100°, and, by interpolation, can be read to about 0.1 degree. It is claimed by the manufacturers that accurate results may be obtained with the indicator to 0.25°, and with the apparatus as a whole to 0.5 degree. From comparative readings made with an ordinary mercury thermometer, it appears that these claims are fully justified. The apparatus installed at Arrowrock was obtained from the Leeds and Northrup Company, of Philadelphia.

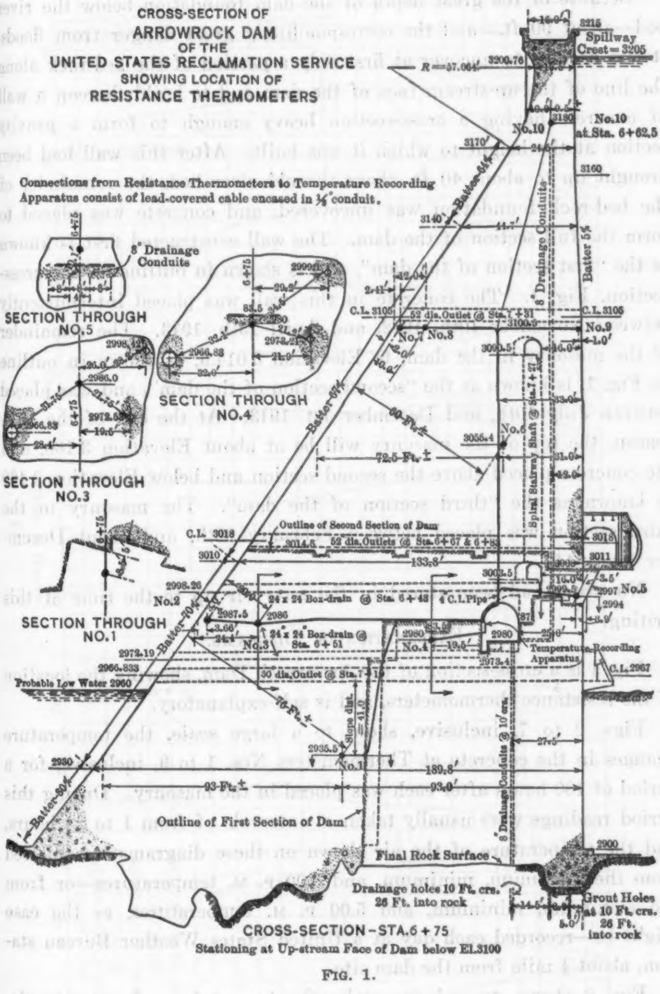
General Method of Placing Thermometers.

The resistance bulbs were placed in one vertical plane, but at various elevations and at different distances from the exposed faces of the dam. Ultimately, there will be one resistance bulb near the top of the dam, one about half way between the original river level and the bed-rock foundation, and eight more at intermediate elevations. When each resistance bulb was placed in the masonry, the lead-covered cable was encased in ½-in. steel tubing and carried to an inspection gallery in the body of the dam, where the switch-board and indicator were installed. When the concrete in the dam was built to the approximate elevation at which it was desired to place a resistance bulb, concreting in that immediate vicinity was usually discontinued temporarily, and the concrete was allowed to set for a few hours. The bulb and conduit were then embedded in a trench, about 6 in. deep, excavated in the concrete, great care being taken to provide a soft, uniform bed for the bulb, and to cover it carefully. Deviation from this practice is mentioned, and further details are given in the following paragraphs.

General Conditions Affecting the Temperature Readings.

The greater part of the masonry in the body of the dam is composed of sand-cement, river sand, river gravel, and river cobbles, mixed in the proportions of about 1 : 2½ : 5 : 2¾, although these proportions have been varied from time to time. Unless otherwise specified, it is to be understood that the cement used in all concrete is sand-cement. In all cases, the sand-cement is the product of grinding, in a tube mill, Portland cement and pulverized granite, mixed in the proportions of 45% of granite, and 55% of Portland cement, by weight. The faces of the dam are composed of a richer concrete than that used in the body, the thickness of the face mix varying with the elevation above bed-rock. In most cases, the cement used in the face mix is composed of about 76% of sand-cement and about 24% of straight Portland cement by volume, and the concrete mix is about 1 : 2 : 4 : 2½.

In some cases, concreting was not carried on continuously directly above the thermometers. An attempt has been made to show this condition on the accompanying diagrams, by giving the depth of concrete directly over the thermometers at frequent intervals of time.



CROSS-SECTION OF
ARROWROCK DAM
OF THE
UNITED STATES RECLAMATION SERVICE
SHOWING LOCATION OF
RESISTANCE THERMOMETERS
Connections from Resistance Thermometers to Temperature Recording Apparatus consist of lead-covered cable encased in ½" conduit.
8" Drainage Conduits
SECTION THROUGH NO.5
SECTION THROUGH NO.4
SECTION THROUGH NO.3
SECTION THROUGH NO.1
Spillway Crest = 3205
3215
3200.76
3180
3170'
3160
3140'
No.10
No.10 at Sta. 6+62.5
Batter 5/6
8" Drainage Conduits
C.L. 3105
52 dia. Outlet @ Sta. 7+31
No.7 No.8
No.9
3099.5'
3055.4
No.6
72.5 Ft.
Spiral Stairway Shaft
Outline of Second Section of Dam
C.L. 3018
3014.5
3010
52 dia. Outlets @ Sta. 6+67 & 6+83
133.8'
3018
3011
3003.5
2998.26
No.2
24 x 24 Box-drain @ Sta. 6+48
4" C.I. Pipe
No.5
2997
2994
2987.5
2986
24 x 24 Box-drain @ Sta. 6+51
No.3
2980
No.4
2980
2973.4
Temperature Recording Apparatus
C.L. 2967
2972.19
2966.833
Probable Low Water 2960
30 dia. Outlet @ Sta. 7+12
2935.5
No.1
189.8
93.0'
27.5
2930
93 Ft.
Batter 80/1
Outline of First Section of Dam
8" Drainage Conduits @ 10' crs.
2900
Final Rock Surface
Drainage holes 10 Ft. crs. 26 Ft. into rock
Grout Holes at 10 Ft. crs. 26 Ft. into rock
CROSS-SECTION - STA. 6 + 75
Stationing at Up-stream Face of Dam below El. 3100

FIG. 1.

The up-stream face of the dam is an east exposure, and the down-stream face is, therefore, a west exposure.

Because of the great depth of the dam foundation below the river bed—about 90 ft.—and the correspondingly great danger from floods, it was decided to uncover at first only a portion of the bed-rock along the line of the up-stream face of the dam and to build thereon a wall of concrete having a cross-section heavy enough to form a gravity section at the height to which it was built. After this wall had been brought up to about 40 ft. above the old river bed, the remainder of the bed-rock foundation was uncovered, and concrete was placed to form the full section of the dam. The wall constructed first is known as the "first section of the dam", and is shown in outline on the cross-section, Fig. 1. The concrete in this wall was placed intermittently between November 10th, 1912, and April 15th, 1913. The remainder of the masonry in the dam, to Elevation 3 014.5, as shown in outline on Fig. 1, is known as the "second section of the dam", and was placed between July 10th, and December 1st, 1913. At the end of the 1914 season, the top of the masonry will be at about Elevation 3 160, and the concrete placed above the second section and below Elevation 3 160 is known as the "third section of the dam". The masonry in the third section was placed between February 26th, and about December 1st, 1914.

No water had been stored in the reservoir up to the time of this writing.

Explanation of Diagrams.

Fig. 1 is a cross-section of the Arrowrock Dam, showing the location of the resistance thermometers, and is self-explanatory.

Figs. 2 to 7, inclusive, show, to a large scale, the temperature changes in the concrete at Thermometers Nos. 1 to 9, inclusive, for a period of 180 hours after each was placed in the masonry. During this period readings were usually taken at intervals of from 1 to 24 hours, and the temperature of the air shown on these diagrams was platted from the maximum, minimum, and 6.00 P. M. temperatures—or from the maximum, minimum, and 5.00 P. M. temperatures, as the case might be—recorded each day at a United States Weather Bureau station, about ½ mile from the dam site.

Fig. 8 shows, to a large scale, the temperature changes in the concrete at Thermometers Nos. 7 and 9 for a period of 48 hours, and

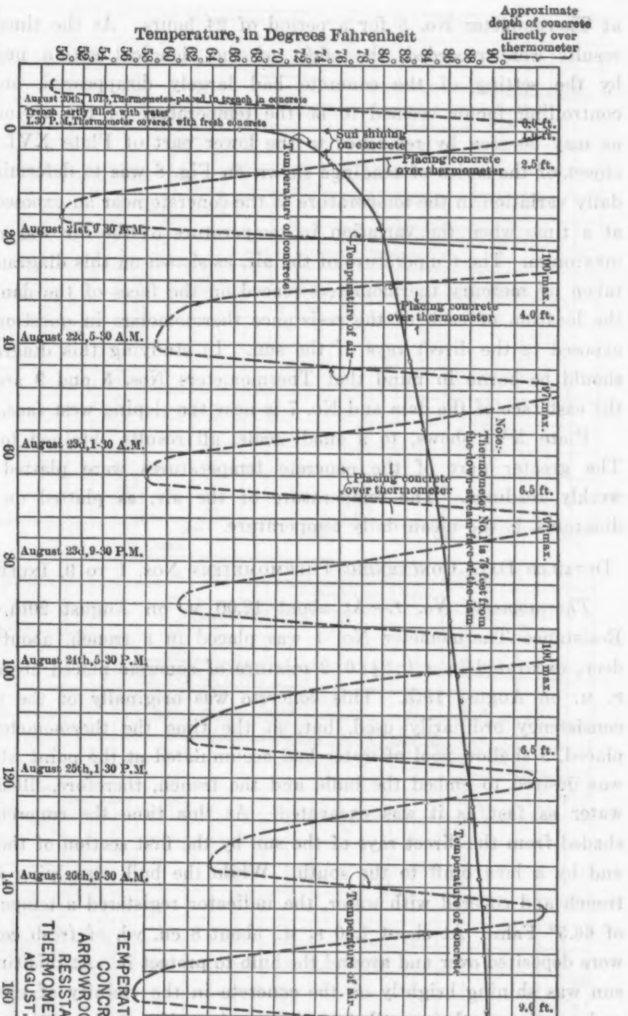

Temperature, in Degrees Fahrenheit
Approximate depth of concrete directly over thermometer
August 20th, 1913, Thermometer placed in trench in concrete
Trench partly filled with water
1.30 P.M. Thermometer covered with fresh concrete
Sun shining on concrete
Placing concrete over thermometer
August 21st, 9-30 A.M.
August 22d, 5-30 A.M.
August 23d, 1-30 A.M.
August 23d, 9-30 P.M.
August 24th, 5-30 P.M.
August 25th, 1-30 P.M.
August 26th, 9-30 A.M.
Temperature of concrete
Temperature of air
Note:- Thermometer No. 1 is 75 feet from the down-stream face of the dam
0.0 ft.
1.0 ft.
2.5 ft.
4.0 ft.
6.5 ft.
9.0 ft.
100 max.
Time, in Hours
TEMPERATURE OF CONCRETE
ARROWROCK DAM
RESISTANCE THERMOMETER NO. 1
AUGUST, 1913

FIG. 2.

at Thermometer No. 5 for a period of 24 hours. At the time these results were recorded, the effect of the chemical action produced by the setting of the concrete had largely disappeared, and the controlling factor seemed to be the temperature of the air outside, as may be seen by reference to the lower part of Plate XVI. The object of the series of readings shown on Fig. 8 was to determine the daily variation in the temperature of the concrete near an exposed face, at a time when the variation in temperature of the air was near a maximum. The temperature of the air, as shown on this diagram, was taken by mercury thermometers placed on the faces of the dam near the location of each of the resistance thermometers in question, and exposed to the direct rays of the sun. In studying this diagram, it should be borne in mind that Thermometers Nos. 5 and 9 are near the east face of the dam and No. 7 is near the sloping west face.

Plate XVI shows, to a small scale, all results obtained to date. The greater part of the concrete temperatures were platted from weekly readings. The temperature of the air, as platted on these diagrams, is the mean daily temperature.

DETAILED DATA CONCERNING THERMOMETERS NOS. 1 TO 9, INCLUSIVE.

Thermometer No. 1.—At about 12.00 M. on August 20th, 1913, Resistance Thermometer No. 1 was placed in a trench, about 6 in. deep, excavated in a 1 : 2½ : 6 : 2 mixture of concrete placed about 3.00 P. M. on August 19th. This concrete was originally of the plastic consistency ordinarily used, but, at the time the thermometer was placed, a shallow pool of water had accumulated at the point where it was desired to embed the bulb, and the trench, therefore, filled with water as fast as it was excavated. At this time the concrete was shaded from the direct rays of the sun by the first section of the dam, and by a lava bluff to the south. While the bulb was lying in the trench and covered with water, the indicator registered a temperature of 66.5° Fahr. At about 1.30 P. M., about 8 cu. yd. of fresh concrete were deposited over and around the bulb to protect it. At this time the sun was shining brightly on the concrete in the vicinity of the bulb, and continued thus until 6.15 P. M. A uniform layer of concrete about 2.5 ft. thick was placed in the immediate vicinity of the bulb between 7.00 and 8.00 P. M. on August 20th. Further details are shown on Fig. 2 and Plate XVI.

PLATE XVI.
TRANS. AM. SOC. CIV. ENGRS.
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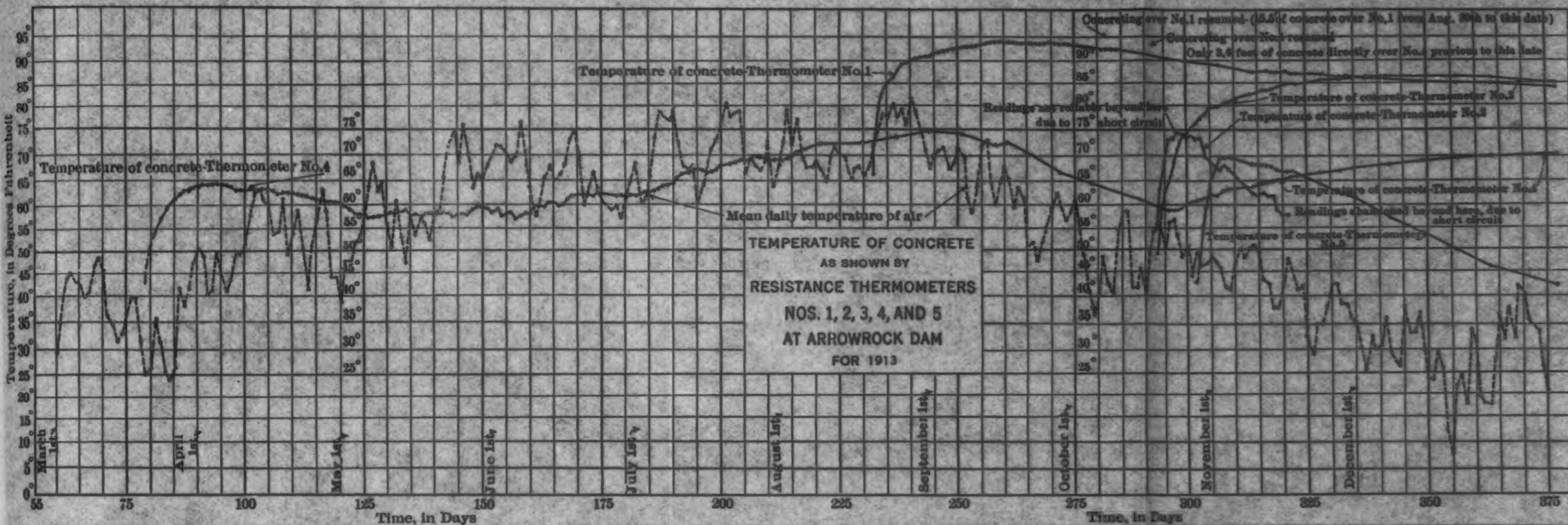
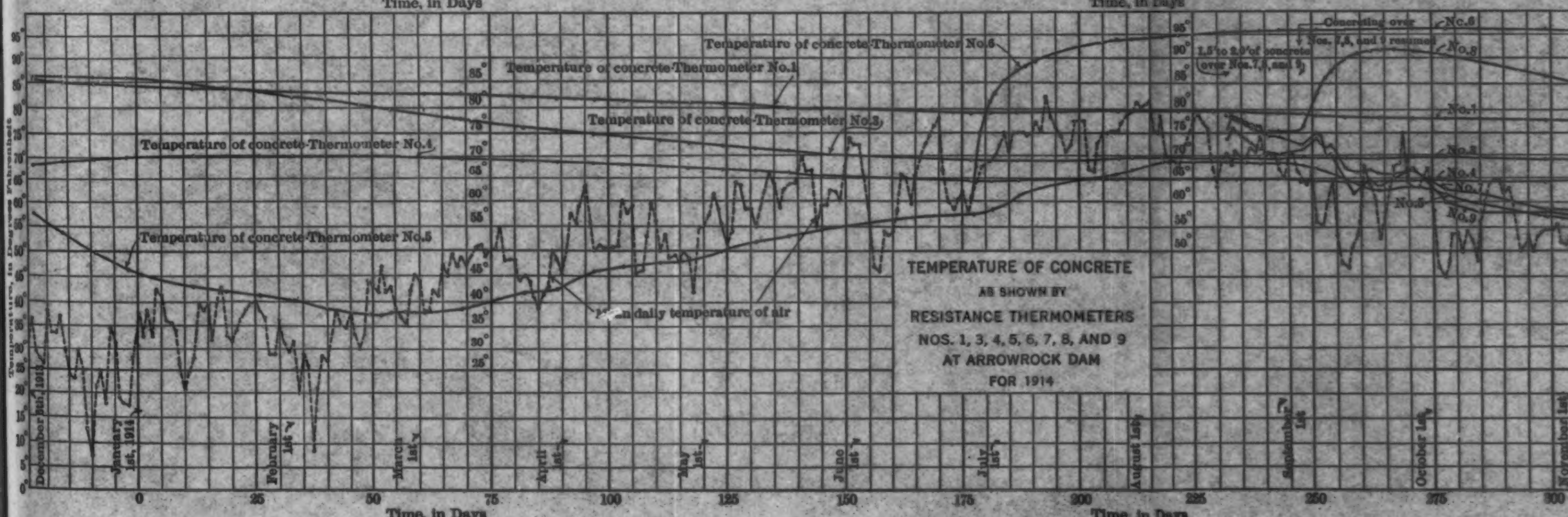
PAUL AND MAYHEW ON
TEMPERATURE CHANGES IN
MASS CONCRETE.





The highest temperature recorded by Thermometer No. 1 was 94.0° Fahr., which reading remained constant from September 20th to 25th, inclusive, at which time there was a mass of 15.5 ft. of concrete directly above the thermometer. Assuming the initial temperature of the concrete as 66.5°, the maximum rise in temperature, due primarily to chemical action, was, therefore, 27.5°, and occurred during a period of 31 days.

Thermometer No. 1 is 76 ft. from the down-stream face of the dam, about 60 ft. above bed-rock, and about 43 ft. from one of the sluicing outlets. The temperature in this outlet was above freezing throughout the winter of 1913-14.

Thermometers Nos. 2 and 3.—At about 3.00 P. M. on October 19th, 1913, Thermometers Nos. 2 and 3 were placed in a trench, about 6 in. deep, excavated in concrete placed about 1.00 A. M. on the same day. Thermometer No. 2 was placed in a 1 : 2 : 4¾ : 1½ mixture of concrete, of plastic consistency. The cement consisted of approximately 78% of sand-cement and 22% of straight Portland cement. Thermometer No. 3 was placed in a 1 : 2½ : 6 : 2 mixture of concrete, although it is probable that this concrete may have contained a small quantity of the face mixture in which No. 2 was embedded. In water content, this concrete, near the surface, at least, was above normal.

Although the greater part of the masonry in the dam was exposed to the direct rays of the sun on October 19th, the portion in which the thermometers were placed was, at that time and during all the winter, in the shadow of the mountains to the south of the dam.

At 3.00 P. M. on October 19th, while the resistance bulbs were lying in the trenches, but before they were covered with concrete, the registration of No. 2 was 57°, and that of No. 3 was 52.8 degrees. The bulbs were then covered with concrete to a depth of about 6 in., to protect them from injury. At about 9.00 A. M. on October 20th, about 2 ft. of the regular 1 : 2½ : 6 : 2 mixture of concrete was placed over Bulb No. 3, and at about 10.00 A. M. No. 3 registered 49.5° and No. 2 55 degrees. Between 10.00 P. M. on October 20th and 2.00 A. M. on October 21st, about 4 ft. of the face mix concrete described above was placed over Bulb No. 2, and the regular mixture of concrete was brought up to a total height of about 4 ft. above Bulb No. 3. Further details are shown on Fig. 3 and Plate XVI.

About October 25th, the temperature readings on Thermometer No. 2 began to show irregularities, and it soon became apparent that there was a partial short circuit. Readings could be taken with a weak battery current, but were undoubtedly not entirely accurate. It seems probable that moisture penetrated the joint between the bulb and the lead cable. It was thought that the continual application of current while the insulation was in a moist condition might permanently injure the insulation; and, as the extent of the inaccuracy in the readings could not be determined, it was decided to discontinue the readings temporarily in the hope that the moisture would ultimately be absorbed by the concrete and the apparatus return to its normal condition. Although some improvement has been shown in the trial readings made from time to time, the results obtained from this thermometer are still unreliable.

The highest temperature recorded by Thermometer No. 2 was 74.5° on October 25th. Assuming the initial temperature of the concrete as 55°, the maximum rise in temperature was therefore 19.5°, and occurred in a period of 5 days. As this thermometer is only 3 ft. from an exposed face, it is probable that the air temperature, which was low at this time, counteracted more or less the heat generated by chemical action.

The highest temperature recorded by Thermometer No. 3 was 86.2°, which reading remained constant from December 4th to 22d, inclusive. Assuming the initial temperature of the concrete as 49.5°, the maximum rise in temperature, due principally to chemical action, was therefore 36.7°, and is the greatest rise recorded by any of the thermometers under consideration. This rise occurred during a period of 45 days. At this time there was a depth of 28.5 ft. of concrete directly over the thermometer.

Thermometers Nos. 2 and 3 are 3.0 ft. and 20.0 ft., respectively, from the down-stream face of the dam.

Thermometer No. 4.—About 5.00 P. M. on March 20th, 1913, Thermometer No. 4, the first one installed in the dam, was embedded about 18 in. in a 2.5-ft. layer of concrete placed during the previous hour or so. After the bulb was placed, concreting in this vicinity was continued until the surface was brought up to about 3.5 ft. above the bulb, at which elevation the surface of the concrete, directly above the bulb, remained for the following 7 months. As will be seen by

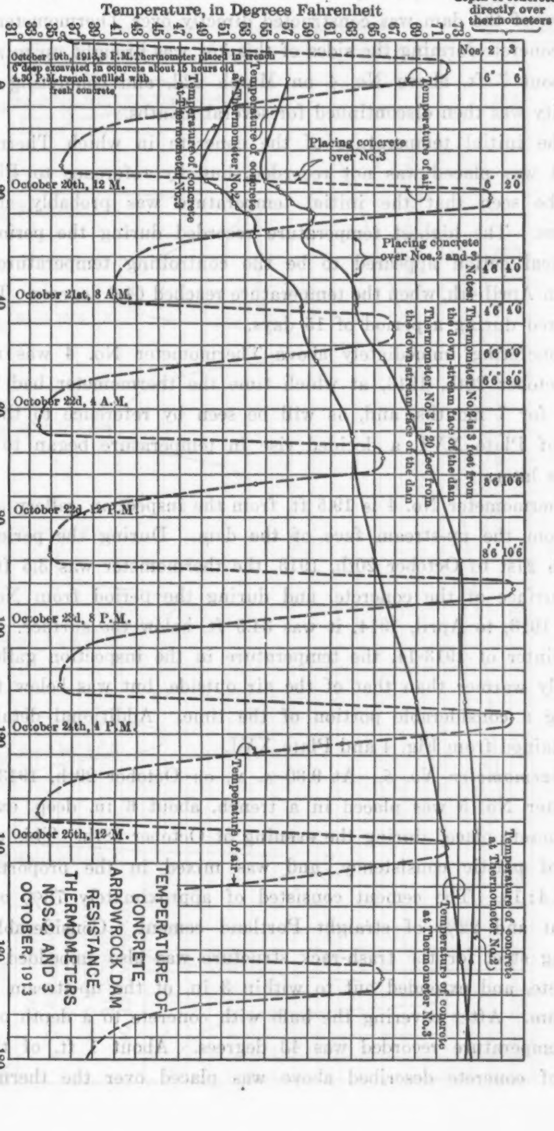

Temperature, in Degrees Fahrenheit
Approximate depth of concrete directly over thermometers
Time, in Hours
October 19th, 1913, 3 P.M. Thermometer placed in trench 6" deep excavated in concrete about 15 hours old
4.30 P.M. trench refilled with fresh concrete
October 20th, 12 M.
October 21st, 8 A.M.
October 22d, 4 A.M.
October 22d, 12 P.M.
October 23d, 8 P.M.
October 24th, 4 P.M.
October 25th, 12 M.
Placing concrete over No.3
Placing concrete over Nos.2 and 3
Temperature of air
Temperature of concrete at Thermometer No.2
Temperature of concrete at Thermometer No.3
Note: Thermometer No.2 is 3 feet from the down-stream face of the dam
Thermometer No.3 is 20 feet from the down-stream face of the dam
TEMPERATURE OF CONCRETE
ARROWROCK DAM
RESISTANCE THERMOMETERS
NOS.2 AND 3
OCTOBER 1913

FIG. 3.

reference to Fig. 1, a key in the surface of the concrete in the first section of the dam was constructed directly over Thermometer No. 4. The concrete forming the sides of this key was brought up to a height of about 7 ft. above No. 4 on March 22d, and concreting in that vicinity was then discontinued for several months.

The initial temperature of the concrete in which Thermometer No. 4 was placed was not recorded, but, by reference to Fig. 4, it will be seen that the initial temperature was probably about 44 degrees. The highest temperature recorded during the period when chemical action appeared to be the controlling temperature factor was on April 4th, when the temperature reached 64.6 degrees. This rise occurred during a period of 15 days.

Concreting immediately above Thermometer No. 4 was resumed on October 20th, 1913, at which time the thermometer had been in place for 7 months, and, as will be seen by reference to the upper half of Plate XVI, a decided rise in temperature began to appear 6 days later.

Thermometer No. 4 is 19.5 ft. from the inspection gallery and 58.5 ft. from the up-stream face of the dam. During the period from March 21st to October 20th, 1913, the thermometer was 3.5 ft. below the surface of the concrete, and during the period from November 15th, 1913, to April, 1914, it was 34.5 ft. below the surface. During the winter of 1913-14, the temperature in the inspection gallery was slightly warmer than that of the air outside, but was below freezing during a considerable portion of the time. Additional details may be obtained from Fig. 4 and Plate XVI.

Thermometer No. 5.—At 9.30 A. M. on October 29th, 1913, Thermometer No. 5 was placed in a trench, about 6 in. deep, excavated in concrete placed during the evening of October 28th. This concrete was of plastic consistency, and was mixed in the proportions of $1:2\frac{3}{4}:4:1\frac{1}{2}$. The cement consisted of approximately 78% of sand-cement and 22% of straight Portland cement. Considerable reinforcing steel for the trash-rack structure was also embedded in this concrete, and extended out to within 3 in. of the up-stream face of the dam. After covering the bulb with concrete to a depth of 6 in., the temperature recorded was 43 degrees. About 1 ft. of the face mix of concrete described above was placed over the thermometer

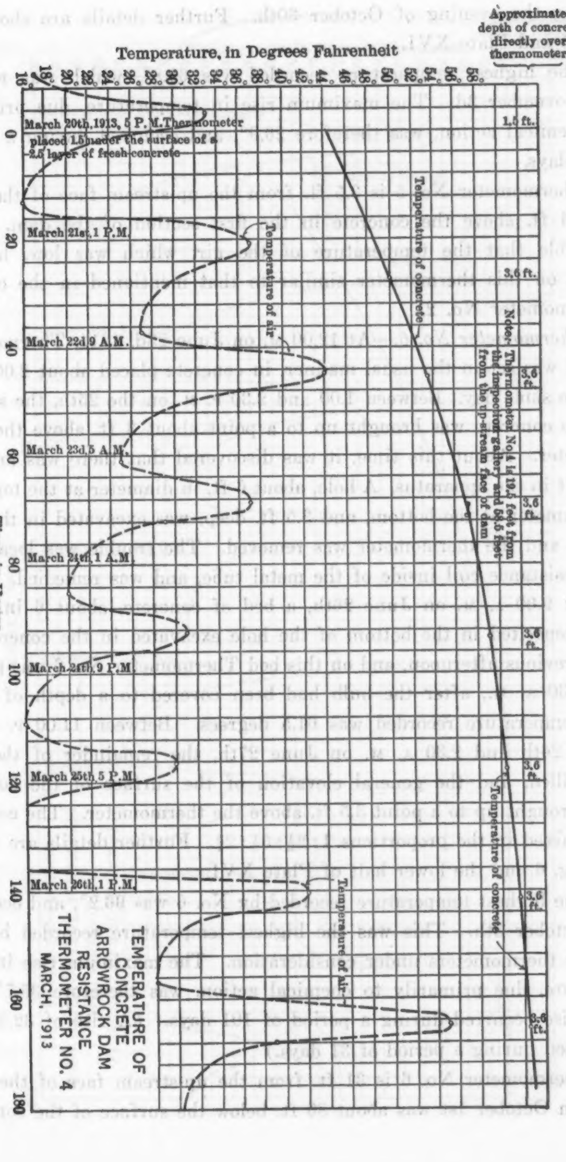

Temperature, in Degrees Fahrenheit
Approximate depth of concrete directly over thermometer
March 20th, 1913, 5 P.M. Thermometer placed 1.5 under the surface of a 2.5 layer of fresh concrete
March 21st, 1 P.M.
March 22d, 9 A.M.
March 23d, 5 A.M.
March 24th, 1 A.M.
March 24th, 9 P.M.
March 25th, 5 P.M.
March 26th, 1 P.M.
Temperature of air
Temperature of concrete
Note: Thermometer No. 4 is 19.5 feet from the inspection gallery and 56.5 feet from the up-stream face of dam
1.5 ft.
3.6 ft.
Time, in Hours
TEMPERATURE OF CONCRETE
ARROWROCK DAM
RESISTANCE THERMOMETER NO. 4
MARCH, 1913

FIG. 4.

during the evening of October 30th. Further details are shown on Fig. 5 and Plate XVI.

The highest temperature recorded was 69.8°, which was reached on November 3d. The maximum rise in temperature, due primarily to chemical action, was therefore 26.9°, and occurred during a period of 5 days.

Thermometer No. 5 is 3.5 ft. from the up-stream face of the dam, and 3 ft. above the concrete in the first section of the dam. It is probable that the temperature of the air, which was low, had an effect on this thermometer similar to that mentioned in the case of Thermometer No. 2.

Thermometer No. 6.—At 12.00 M. on June 23d, 1914, Thermometer No. 6 was set in the usual manner, in concrete placed about 2.00 A. M. on the same day. Between 1.00 and 2.30 P. M. on the 25th, the surface of the concrete was brought up to a point about 3 ft. above the thermometer. About this time, it was discovered that there was an open circuit in the apparatus. A hole, about 6 ft. in diameter at the top, 2 ft. in diameter at the bottom, and 3.5 ft. deep, was excavated in the concrete, and the thermometer was removed. The trouble was located in the resistance coil inside of the metal tube, and was remedied.

At 9.00 A. M. on June 26th, a bed of concrete, about 6 in. deep, was deposited in the bottom of the hole excavated in the concrete on the previous afternoon, and on this bed Thermometer No. 6 was placed. At 9.30 A. M., after the bulb had been covered to a depth of 6 in., the temperature recorded was 61.5 degrees. Between 11.00 P. M. on June 26th and 2.30 A. M. on June 27th, the remainder of the hole was filled, and the general elevation of the surface of the concrete was brought up to a point 3.5 ft. above the thermometer. The concrete was mixed in the proportions $1:2\frac{1}{2}:5\frac{1}{2}:2\frac{3}{4}$. Further details are shown on Fig. 6 and the lower half of Plate XVI.

The highest temperature recorded by No. 6 was 96.2°, and occurred on October 5th. This was the highest temperature recorded by any of the thermometers under consideration. The maximum rise in temperature, due primarily to chemical action, was therefore 35.7°, and this rise occurred during a period of 101 days. (A rise of 32.2° was recorded during a period of 31 days.)

Thermometer No. 6 is 31 ft. from the up-stream face of the dam, and on October 1st was about 80 ft. below the surface of the concrete.

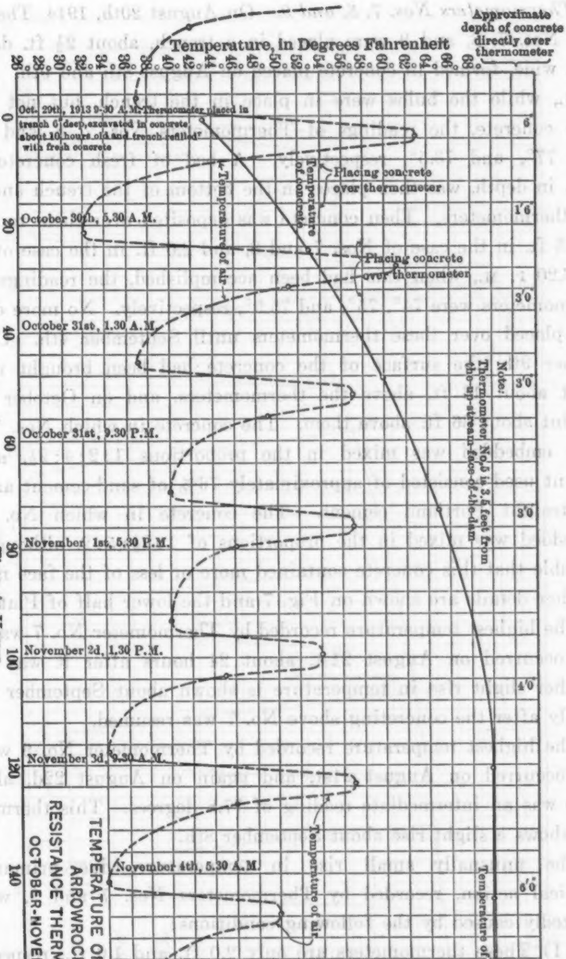

Temperature, in Degrees Fahrenheit
26 28 30 32 34 36 38 40 42 44 46 48 50 52 54 56 58 60 62 64 66 68
Approximate depth of concrete directly over thermometer
Oct. 29th, 1913 9-30 A.M. Thermometer placed in trench 6" deep, excavated in concrete, about 10 hours old and trench refilled with fresh concrete
Temperature of concrete
Temperature of air
Placing concrete over thermometer
October 30th, 5.30 A.M.
Placing concrete over thermometer
October 31st, 1.30 A.M.
October 31st, 9.30 P.M.
November 1st, 5.30 P.M.
November 2d, 1.30 P.M.
November 3d, 9.30 A.M.
November 4th, 5.30 A.M.
Note: Thermometer No. 5 is 3.5 feet from the up-stream face of the dam
Temperature of concrete
Temperature of air
6" 1'6" 3'0" 3'0" 3'0" 4'0" 6'0"
0 20 40 60 80 100 120 140 160 180
Time, in Hours
TEMPERATURE OF CONCRETE
ARROWROCK DAM
RESISTANCE THERMOMETER NO. 5
OCTOBER-NOVEMBER, 1913

FIG. 5.

Thermometers Nos. 7, 8, and 9.—On August 20th, 1914, Thermometers Nos. 7, 8, and 9 were placed in a trench, about 2½ ft. deep and 3 ft. wide, formed in concrete placed on August 8th and 9th. At 1.30 P. M., while the bulbs were in place in the trench, but not covered with concrete, the readings of Thermometers Nos. 7, 8, and 9 were 77°, 77°, and 73.5°, respectively. A bed of fresh concrete, about 6 in. in depth, was then placed in the bottom of the trench and under the thermometers. Then concrete was deposited over them to a depth of 1.5 ft. in the case of Nos. 7 and 8, and 2.0 ft. in the case of No. 9. At 3.20 P. M., when this had been accomplished, the readings of the thermometers were 74°, 75°, and 75.5°, respectively. No more concrete was placed over these thermometers until September 4th. On September 9th, the surface of the concrete had been brought up to a point about 13 ft. above the thermometers, and on October 1st, to a point about 36 ft. above them. The concrete in which Nos. 7 and 9 were embedded was mixed in the proportions 1:2:4:2½, and the cement used consisted of approximately 76% of sand-cement and 24% of straight Portland cement. The concrete in which No. 8 was embedded was mixed in the proportions of 1:2½:5:2, although it is probable that this concrete contained more or less of the face mixture. Further details are shown on Fig. 7 and the lower half of Plate XVI.

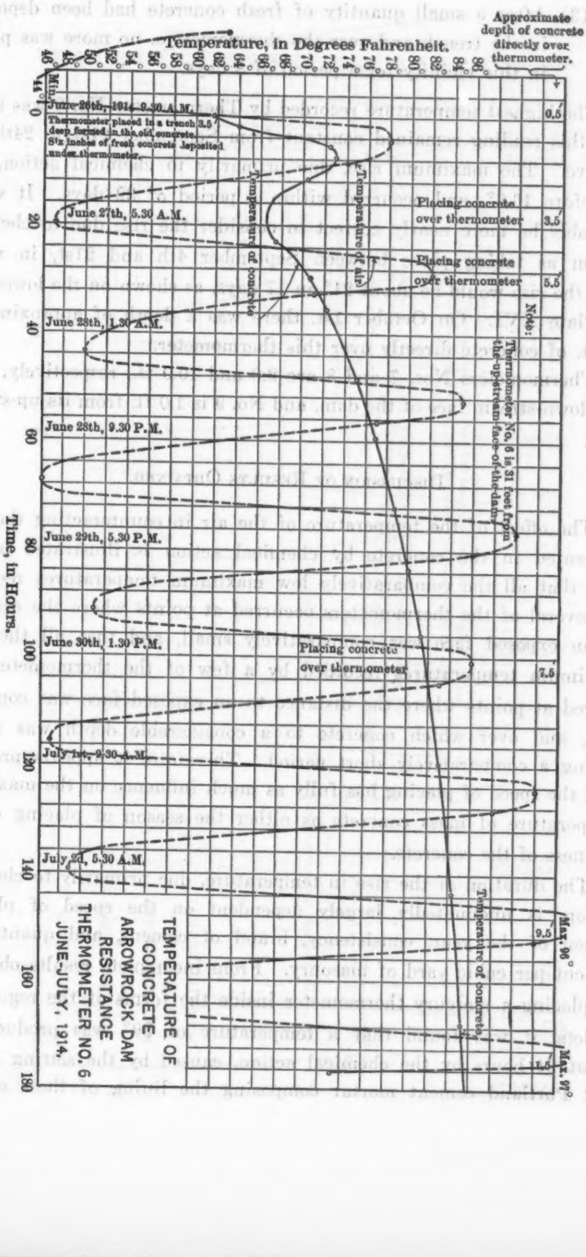
The highest temperature recorded by Thermometer No. 7 was 77.6°, and occurred on August 21st, about 24 hours after it was placed. Another slight rise in temperature is shown about September 8th, or shortly after the concreting above No. 7 was resumed.

The highest temperature recorded by Thermometer No. 9 was 78°, and occurred on August 21st, and again on August 22d, although there was an intermediate reading of 77.8 degrees. This thermometer also shows a slight rise about September 8th.

The unusually small rise in temperature, due primarily to chemical action, recorded by Thermometers Nos. 7 and 9, was undoubtedly caused by the following conditions:

(1) These thermometers are only 2.0 ft. and 1.0 ft., respectively, from the faces of the dam, and were covered to a depth of only 1.5 ft. and 2.0 ft., respectively.

(2) The concrete forming the sides of the trench in which the thermometers were placed was deposited 11 or 12 days previous to the time they were put in position.



Temperature, in Degrees Fahrenheit.
Approximate depth of concrete directly over thermometer.
June 26th, 1914, 9.30 A.M.
Thermometer placed in a trench 3.5' deep formed in the old concrete. 8½ inches of fresh concrete deposited under thermometer.
June 27th, 5.30 A.M.
June 28th, 1.30 A.M.
June 28th, 9.30 P.M.
June 29th, 5.30 P.M.
June 30th, 1.30 P.M.
July 1st, 9.30 A.M.
July 2d, 5.30 A.M.
Temperature of concrete
Temperature of air
Placing concrete over thermometer
Note: Thermometer No. 6 is 31 feet from the up-stream face of the dam.
0.5'
3.5'
5.5'
7.5'
9.5'
Max. 96°
Max. 97°
Min. 44°
Time, in Hours.
TEMPERATURE OF CONCRETE ARROWROCK DAM RESISTANCE THERMOMETER NO. 6 JUNE-JULY, 1914.

FIG. 6.

(3) After a small quantity of fresh concrete had been deposited in the trench and over the thermometers, no more was placed in this vicinity for a period of 8 days.

The highest temperature recorded by Thermometer No. 8 was 91.6°, and this reading remained constant from September 21st to 24th, inclusive. The maximum rise, due primarily to chemical action, was therefore 16.6°, and occurred within a period of 32 days. It would probably be more nearly correct to consider the rise due to chemical action as taking place between September 4th and 21st, in which case the rise would be about 21° in 17 days, as shown on the lower half of Plate XVI. On October 1st, there was a depth of approximately 35 ft. of concrete directly over this thermometer.

Thermometers Nos. 7 and 8 are 2.0 and 10.0 ft., respectively, from the down-stream face of the dam, and No. 9 is 1.0 ft. from its up-stream face.

Discussion of Results Obtained.

The effect of the temperature of the air in counteracting the heat generated in the concrete by chemical action is illustrated by the fact that all the comparatively low maximum temperatures recorded by several of the thermometers occurred at points where the distance to an exposed face was comparatively small, and that all the high maximum temperatures recorded by a few of the thermometers occurred at points where the distance to an exposed face was considerable, and over which concrete to a considerable depth was placed during a comparatively short period. Therefore, it appears probable that the speed of placing has fully as much influence on the maximum temperature of mass concrete as either the season of placing or the richness of the concrete.

The duration of the rise in temperature, due primarily to chemical action, is undoubtedly largely dependent on the speed of placing, season of the year, consistency, brand of cement, and quantity of cement per cubic yard of masonry. From incomplete results obtained by placing a mercury thermometer inside the forms of the regulating outlets, it was found that a temperature of 99° was produced in about 24 hours by the chemical action, caused by the setting of the 1:2 Portland cement mortar composing the lining of these outlets.

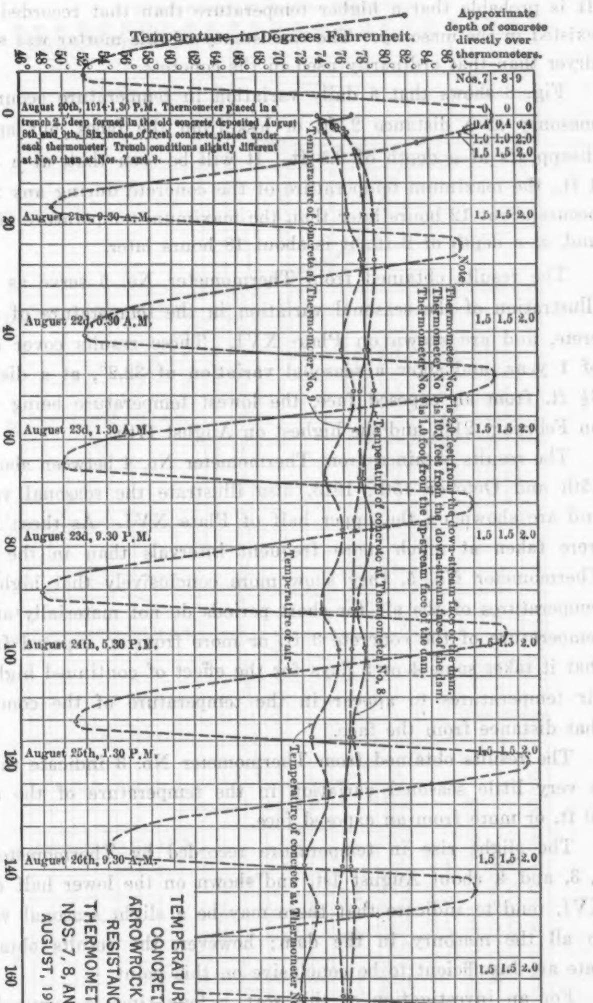


FIG. 7.

TEMPERATURE OF CONCRETE ARROWROCK DAM RESISTANCE THERMOMETERS NOS. 7, 8, AND 9 AUGUST, 1914.

It is probable that a higher temperature than that recorded actually existed in the masonry. The consistency of this mortar was somewhat dryer than that ordinarily used in the concrete.

Fig. 8 shows that a daily variation in temperature occurs in the masonry at a distance 2 ft. or less from the faces, but apparently disappears at a depth of 3.5 ft. It will be seen that, at a depth of 1 ft., the maximum temperature of the concrete during any 24 hours occurs about 12 hours later than the maximum temperature of the air, and, at a depth of 2 ft., it is about 18 hours later.

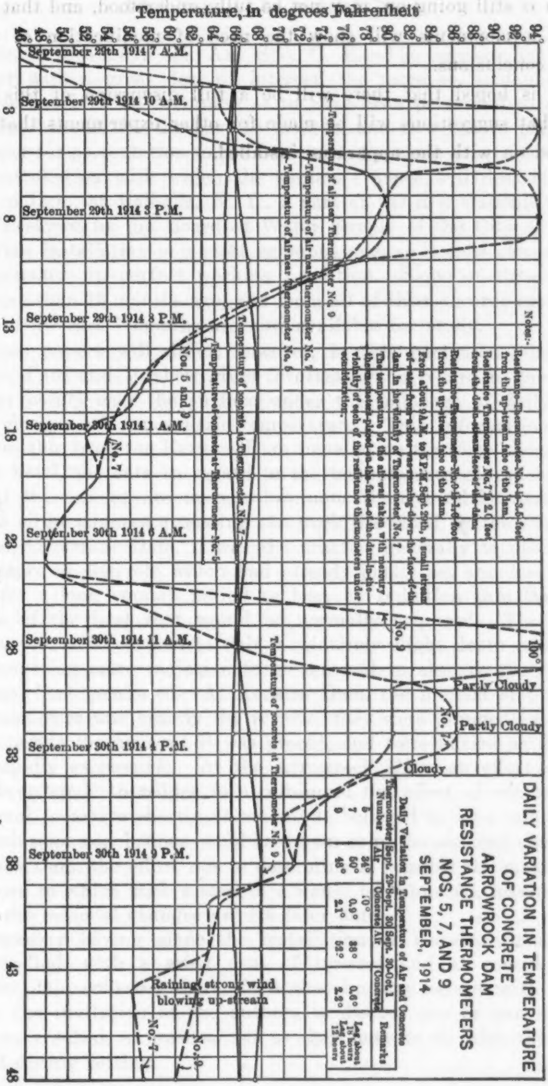
The results obtained from Thermometer No. 5 serve as the best illustration of the seasonal variation in the temperature of the concrete, and are shown on Plate XVI. These results cover a period of 1 year, and show a seasonal variation of 32.2°, at a distance of 3½ ft. from an exposed face, the lowest temperature being recorded on February 21st, and the highest on August 17th.

The results obtained from Thermometer No. 4 between about April 15th and October 15th, 1913, also illustrate the seasonal variation, and are shown on the upper half of Plate XVI. As these readings were taken at much more frequent intervals than in the case of Thermometer No. 5, they show more conclusively that high or low temperatures of the air for short periods do not materially affect the temperature of the concrete 3 ft. or more from an exposed face, and that it takes some 4 or 5 days for the effect of continued high or low air temperatures to appear in the temperature of the concrete, at that distance from the face.

The results obtained from Thermometer No. 3 indicate that there is very little seasonal variation in the temperature of the concrete 20 ft. or more from an exposed face.

The slight rise in temperature recorded by Thermometers Nos. 1, 3, and 4, about August 1st, and shown on the lower half of Plate XVI, tend to indicate that there may be a slight seasonal variation in all the masonry in the dam; however, the results obtained to date are insufficient to be conclusive on this point.

For an investigation of this sort, a long time is required before the drawing of many definite conclusions is warranted. Most of the opinions expressed in this paper seem to be warranted by the facts, but it is realized that the effect of chemical action in the concrete,



Time, in Hours

FIG. 8.

which is still going on, may not be fully understood, and that possibly later developments will show the need of modification of some of these conclusions.

It is hoped that there will be a full discussion of this subject, and that suggestions will be made for other experiments that may be carried on with the apparatus installed.

DISCUSSION

GEORGE T. SEABURY,* M. AM. SOC. C. E.—The speaker has read this paper with a great deal of interest, the more so, undoubtedly, because he has been so fortunate as to have been permitted to study this subject himself for the past 18 months, and now has in preparation a paper for presentation to the Society on the same general subject. Mr. Seabury.

The installation with which the speaker's paper will deal is that at Kensico Dam, at Valhalla, N. Y., a part of the new Catskill water system, developed by the Board of Water Supply of the City of New York. This installation is a most generous one, as it now consists of 45 thermometers in perfect working condition. Some of these were placed more than 19 months ago, and from all of them records covering a period of at least 7 months are now available for study.

The two papers will differ, however, in this particular, the one by Messrs. Paul and Mayhew gives information in regard to the fluctuation at points near the surface under the influence of daily and seasonal changes of atmospheric temperature. This information will not be available from the Kensico Dam installation for some time. On the other hand, the data in regard to the temperatures caused by the setting of the cement are more voluminous than could be available under the circumstances attending the work described by the authors.

At the Arrowrock Dam, it was the practice generally to place the thermometers in concrete which had already partly set and had been exposed for a long enough period to lose, by radiation into the air, a portion of the heat that would be normally acquired. The thermometers, too, were covered to only a relatively slight depth in many cases, permitting more radiation to go on, and so obscure the record of the total heat generated. At Kensico Dam, the method of placing the thermometers was exactly the reverse, they were plunged into the plastic concrete the moment it was placed, and were buried as deeply and as rapidly as possible. At the Arrowrock Dam, an effort seems to have been made to reduce to a minimum the effect of the setting cement, and to arrive shortly at what was believed to be a condition from which some conclusions could be drawn as to the seasonal changes. At Kensico Dam, the effort was to determine if possible the maximum temperature to which such a structure would be subjected, leaving the study of the seasonal changes until a later date.

The speaker having made the installation at Kensico Dam, and having studied, with a great deal of interest and at considerable length, the data collected there, feels that he may accept with some propriety the invitation of the authors to discuss one or two of the conclusions reached, at least so far as they have to do with the heat generated during setting.

* New York City.

Mr. Seabury.

The authors state: "it appears probable that the speed of placing has fully as much influence on the maximum temperature of mass concrete as either the season of placing or the richness of the concrete". The experience gained at Kensico leads the speaker to differ on this point. It was clearly demonstrated on that work that, provided the initial heat acquired in setting be not dissipated through a slight amount of cover, the rise in temperature is a constant for any given mix, irrespective of the season of placing. It appears that the speed of placing has a great deal to do with the rise of temperature, so far as reducing it is concerned; but, on the other hand, it seems that, with a sufficient amount of cover, the rise will be a constant for any given mix. From this it follows that the season of placing, or rather the initial temperature of the aggregates, has much to do with the maximum temperature reached.

Kensico Dam is a gravity structure, about 27% of its mass being of large stone; the remainder is of concrete, in the proportion of 1 part cement to 9 parts aggregate, and with this mixture a uniform rise of temperature of 40° was apparently well established. This resulted, in the summer, in a maximum temperature well above 100 degrees. The highest temperature observed—and that which it is believed is probably approximately the maximum in any part of the structure—was 118.5° Fahr.; but—and this is the point which seems to be of the most value—this maximum temperature, as the authors have shown, is not reached until long after the concrete has attained its final size and shape.

The authors conclude, too, that "the duration of the rise in temperature, due primarily to chemical action, is undoubtedly largely dependent on the speed of placing, season of the year, consistency, brand of cement, and quantity of cement per cubic yard of masonry." If they had omitted the words "the duration of" so that the sentence would read: "the rise in temperature, due primarily to the chemical action, is undoubtedly largely dependent on the speed of placing, season of the year, consistency, brand of cement, and quantity of cement per cubic yard of masonry", the conclusion would then have been in almost exact accord with that drawn from the Kensico installation, with the one exception as to the season of the year. To conclude that the duration of the rise is also due to these factors is a new thought to the speaker, and was not clearly demonstrated at Kensico, beyond the fact that the normal rise was inhibited in both duration and amount by radiation when the point under observation was near an exposed surface, whether that was the face of the dam or the masonry as left uncompleted on top.

Conclusion (1), that "Large bodies of concrete deposited rapidly during a summer season develop a temperature of from 90 to 95° within a period of about 30 days, and maintain nearly that temperature

for several months", appears to be true only so far as it goes. The speaker would express it as follows: Large bodies of concrete deposited rapidly develop a temperature a definite amount greater than the initial temperature of the aggregate, probably dependent almost entirely, if deposited rapidly enough to minimize radiation, on the quantity of cement to the unit of the resulting mass. It is the speaker's opinion that, with rich mixtures, this maximum temperature may be even 135 or 140° Fahr. In the lock walls at Panama, a temperature of about 135° was observed, if the speaker remembers correctly; and, in a much smaller mass of rich concrete placed in one of the shafts for the Board of Water Supply, one of 136° was observed. The time required for the acquisition of the maximum temperature at Kensico Dam was found to be somewhat less than 30 days. Mr. Seabury.

The speaker feels, as do the authors, that, as time goes on, these installations will prove fruitful of more and more definite conclusions, and that the results to be obtained from a study of them warrant a full and early presentation of all the facts of installation, which, it is sincerely hoped, will be followed by papers from time to time, bringing the new data before the Society.

Mr. Immediato has asked whether or not there was observed at Kensico Dam any evidence that the heat generated by the setting cement varies with the brand of cement used, the quantity of water used, the quantity of cement, or the quantity of free lime in the cement. There was no strong evidence to enable one to answer positively any of these questions. It is the speaker's belief that the quantity of cement per unit of resulting mass does affect strongly the rise in temperature; but, as all the thermometers were set in concrete having the same proportions, no evidence was obtained. Other installations, however, in concrete of richer mix prove this apparently beyond question, as pointed out previously.

As to whether or not the brand of cement affects the quantity of heat generated, the evidence again is very meager. There was one place in the Kensico Dam where two thermometers were set about 5 ft. from each other, and under conditions which were thought to be absolutely identical, with one exception. After each had been covered by about 2 ft. of concrete made with Atlas cement, a few batches of concrete made with Alsen cement were brought to the dam, and because it was desired to have the thermometers covered deeply and quickly, this was added to the existing cover. The concrete made with the Alsen cement, however, was kept by itself and placed over only one of the thermometers. Within the first 6 hours after being placed, the thermometer which was nearer the Alsen cement did not rise as high as the one which was entirely surrounded by Atlas cement, and which had no Alsen near it; but, subsequently, they reached almost exactly the same temperature. The evidence is, then, that the brand of cement

Mr. Seabury.

appears to make some difference at a certain stage, but not ultimately.

It is known from laboratory experiments that the heat generated and measured during mixing is different for different brands of cement, and it would seem probable that that difference would continue throughout the entire action of the cement, but no evidence was obtained at Kensico Dam to prove it.

In regard to the question brought up by Mr. Williams, as to the frequent calibration of the thermometers, it may be said that all the instruments were carefully calibrated to determine their individual corrections before they were installed. As some of them are now 100 ft. or more deep in the masonry, it is, of course, impossible to calibrate them again, and yet there is a sort of check obtained on them. There are several thermometers showing almost constant temperatures. Two of these are in the rock beneath the dam, and they have not exhibited any considerable change during the last 9 months. Certain others too, which are well protected, are exhibiting a very slow and uniform change of temperature. When, therefore, there is obtained on any thermometer a reading which does not "look right" (and that is the easiest way to express it), a reading is obtained on one of these constant thermometers, and if this reading also does not "look right", it is evident that there is trouble. If, however, the constant thermometers read in accordance with their past history, there is generally no good reason to doubt that the other is incorrect until more readings are obtained.

One thermometer may go wrong, although others connected with the same switch-board do not, but it seems to be true that the sensitive coil—the part which is in the masonry—does not vary. It has been found, however, that the parts which are in the switch-board may vary because of a number of conditions, such as short-circuits, grounds, leakages, etc., and when there is trouble, or when, in a reading, anything looks wrong, attention is always directed to the switch-board, and after a sufficiently careful hunt, the trouble is almost always to be detected there.

Mr. W. F. Smith.

WILSON FITCH SMITH,* M. AM. SOC. C. E.—This paper is of interest to the speaker because it describes an experiment which is quite similar to one that he is conducting at the present time, to determine the internal temperatures of the masonry at Kensico Dam. It is also of interest to learn that at both Arrowrock and Kensico seemingly similar difficulties have been experienced; they have been overcome in much the same way, and, as far as results have been obtained, have led to similar conclusions.

The use of electrical resistance coils for determining temperatures in masonry is not a new idea. There is a description of an installation

* New York City.

for such purpose at the Boonton Dam, in a discussion* by Thaddeus Merriman, M. Am. Soc. C. E., and there are records of a number of similar installations. In the earlier instruments, the balance of the resistances in the Wheatstone bridge was determined by a telephone receiver in the circuit, hence the term "thermophone". More recently, the telephone has been replaced by a galvanometer, and it would seem that the term "electrical resistance thermometer", used in the paper, is more correct. Earlier experiments have proved somewhat disappointing, because of the more or less complete failure, after a while, of the coils or cables, due to imperfect insulation. Manufacturers of recent apparatus have benefited by the experience of the past, and with the lead-sheathed cable and more carefully constructed joints there is every hope that in these present installations results of considerable value, and extending over a period of years, may be obtained. Mr. W. F. Smith.

The installation at Kensico Dam may be described briefly as follows:

The dam, in Westchester County, is being built by the City of New York to form a storage reservoir for the Catskill water supply. It will be about 1 850 ft. long on top; its maximum height will be 307 ft.; its width under the copings will be 28 ft., and it will contain about 900 000 cu. yd. It is being built of Cyclopean masonry, the water side being faced with concrete blocks, and the down-stream side with cut stone. It is divided into sections by twenty-two transverse contraction joints which are about 79 ft. apart and serve as construction joints.

The concrete is mixed in the proportion of 1:3:6, Atlas and Alsen brands of cement being used. The concrete is mixed in gravity mixers, and is handled rapidly in 2-yd. buckets, so that the elapsed time between mixing and placing is very short, usually not more than 5 or 6 min.

The portion of the dam chosen for the installation of the electric thermometers is near the center, and was built comparatively rapidly. There are forty-seven sensitive coils, and these are connected by cables to three switch-boards at convenient points in the inspection galleries. These switch-boards afford a ready means of attaching the recording instrument to the cables. The apparatus was made by Mr. Henry E. Warren, of Ashland, Mass., and has proved to be very satisfactory.

The speed with which this portion of the dam was built is indicated by the following: In the fall of 1913, a height of 35 ft. was placed, and in that height 15 thermometers were installed. During the following summer, the masonry was carried to the top—a distance of 165 ft.—in 6 months, and this part contains the remaining 32 thermometers.

* "The Effect of Temperature Changes on Masonry", by the late Charles S. Gowen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXI, p. 417.

Mr. W. F. Smith.

The thermometers were placed in two planes, one near a contraction joint, and the other midway between two contraction joints, or in the center of a section. In each plane the thermometers are placed in corresponding positions, and at various elevations, near the upstream and down-stream faces, at equal distances from both faces, and at the middle line between those faces.

Although some of the thermometers have been in place for 18 months, and others for more than 1 year, and fairly continuous readings on all of them have been obtained, it is too early to draw any definite conclusions as to the variations of temperature in the masonry due to seasonal changes, for the reason that, even at the present time, most of the thermometers are influenced by the heat developed by the setting of the cement.

Thus far, the observations can only indicate some of the phenomena attendant on the generation of heat due to the setting of the cement and the subsequent cooling off, the more important of which are as follows:

A.—A number of the locations, well away from the face of the dam, indicated that, for this particular mixture of concrete, the rise of temperature was about 40° Fahr., and this seemed to be irrespective of the temperature at which the concrete was placed. The maximum temperatures, when undisturbed by other conditions, were reached in about 15 days—some points obtained their maximum temperature in 5 days and others not for 95 days.

B.—The increase in temperature due to the setting of the cement in the concrete began as soon as the latter was placed, and increased at the rate of about 1° per hour, for 4 or 5 hours, when the rate gradually increased to from 8 to 10° per hour, for an hour or so, and then suddenly dropped to about ½° per hour for a considerable time. The period of most rapid heat development corresponds with that of the final set of cement under ordinary conditions. Many observations indicate a rise in temperature of from 25 to 30° Fahr. after this period. This fact suggests the question: What is the effect on the strength of a large mass of masonry rapidly placed, if in its interior a temperature of from 25 to 30° Fahr. is developed after the cement has obtained its final set and the masonry its solidified form? Is the increase in size which results from this heating sufficient to develop disruptive strains in the fresh masonry beyond its young strength? No surface cracks which would corroborate this idea have been observed, but it is possible that the results of this initial heating would not be manifest to the eye and yet be sufficiently serious to affect the final strength of the masonry. The speaker feels that this is a matter which should receive further consideration.

C.—Another observed fact is the considerable difference between the shady and sunny side of the dam. The dam lies approximately

east and west, with the water side to the north, and the down-stream or exposed face to the south. Thermometers at a depth of 8 ft. from the down-stream face record, at the present time, temperatures which are from 15 to 22° higher than those in a corresponding position 8 ft. from the up-stream face. Mr. W. F. Smith.

George T. Seabury, M. Am. Soc. C. E., who had personal charge of the installation at Kensico Dam, expects to present to the Society all the details of the observations so far obtained at Kensico Dam in a paper in the near future. He is much more familiar with the details than the speaker, and, therefore, is better qualified to discuss this paper in detail.

Mr. Immediato.

G. IMMEDIATO,* ASSOC. M. AM. SOC. C. E.—The speaker has no definite knowledge on the subject, but the following questions are suggested, and may probably have some bearing on the differences in temperature obtained in the two dams:

1. Does the brand of cement affect the intermediate temperatures, at different depths from exposed surfaces, during setting, as well as the final temperature?

2. What effect, if any, is produced by the presence of free lime in the cement?

3. Does the density of the concrete have any bearing on the subject?

4. The quantity of water used in the mixture must surely affect the temperatures.

Mr. Williams.

GARDNER S. WILLIAMS,† M. AM. SOC. C. E.—In the use of thermophones, it is desirable occasionally to calibrate the instruments by comparison with the more reliable thermometer, or, at least, this was formerly so. The speaker has not done any work with the thermophone for a number of years, but during the Nineties, use was made of it in measuring temperatures in the Detroit River, and it was found necessary to check the instrument with a mercury thermometer every day, appreciable variation appearing at times. Of course, where great changes of temperature are involved, the variations would not amount to much, but where it is undertaken to determine just what the change is in a few hours, or from one day to another, there is a possibility, perhaps, of serious errors if they are not guarded against.

In the work to which reference is made, it was not ascertained whether the difficulty was in the recording mechanism or in the coil itself. Probably it was in the recording mechanism, as it is hard to conceive how the thermo-electric coil could vary its indications from day to day.

There is another suggestion which might be offered to those investigating the subject of this paper, and that is, that pressures due to

* New York City.

† Ann Arbor, Mich.

Mr. Williams. the superincumbent masses of concrete may have some effect on the indicated temperatures.

This question of the effect of temperature changes on the condition of the mass of material is probably related to that of Poisson's ratio, that is, the ratio between the stresses which are set up opposite to the applied force and those at right angles thereto, and it may happen that very great changes of temperature can take place under some conditions without any serious effect, whereas, under others, a much less extensive change would produce effects which would be dangerous.

That all material exists under stress must not be forgotten, and although that fact may have no direct bearing on the temperature of concrete, it may on the temperature in natural rock, for it is well known that the walls of a quarry gradually approach each other as the rock is removed from it. That is probably due to the gradual change of stress in the material as pressure is removed on one side, and it must be accompanied by temperature changes. That action seems to go on for a long time; in fact, it was noted at Cornell University, during a period of 2 or 3 years, from measuring the width of the canal, cut in the shale rock, at the Hydraulic Laboratory. The length of the weir across the canal was measured frequently, and, in comparing results, it was discovered that it decreased measurably as time went on. The weir was approximately 16 ft. long, the ends being marked by angle irons set in the concrete which rested against the rock. The observed change, from the fall of 1899 to the summer of 1901, was about 4 mm. The excavation was made about 2 years prior to the former date.

Mr. W. M. Smith. WALTER M. SMITH,* M. AM. SOC. C. E. (by letter).—The writer has read this paper with much interest. It is very valuable, as the Engineering Profession is urgently in need of additional data on the setting temperatures in concrete masonry.

For years the writer has felt that the practice of many engineers in assuming that concrete arches will set at a mean temperature, and that a variation of the same, or nearly the same, amount above and below this mean is to be expected in the future, is not good practice. From evidence in his possession, he feels confident that, if rich in cement, concrete, even in midwinter, if deposited in large masses and covered by wooden forms, will set very often at a temperature of more than 100 degrees.

The writer, therefore, believes that Conclusion (1) should be modified somewhat in its applications. It is stated by the authors that the bulk of the masonry is composed of 1 : $2\frac{1}{2}$: 5 : $2\frac{3}{4}$, and that the sand-cement is composed of 45% of granite and 55% of Portland

* Dayton, Ohio.

cement. The proportion of Portland cement in the loose materials by volume, therefore, was only 5% in the bulk of the masonry and 7% in the facing. In the ordinary 1:3:6 concrete mixture, there is 10% of cement by volume loose, and in the 1:2:4 mixture, generally used for reinforced concrete, the cement amounts to 14.3 per cent. The writer is convinced that had the cement component been higher in this masonry, the setting temperature would have been materially higher. The qualification should be made, therefore, that this rise is to be expected in concrete very lean in cement. Another qualification which the writer believes should be made is concerning the time required for the concrete to reach its maximum temperature, which is stated to be 30 days. The writer is of the opinion that this is much too long, except for a very lean mixture. Mr. W. M. Smith.

In the spillway bridge built by the Board of Water Supply, of New York City, at the Ashokan Reservoir, several thermophones were placed at various points in the arch to record the temperature changes. Three were placed at the crown, five at a point about midway between the crown and the abutment, and three at the abutment. The arch was 3 ft. 4 in. thick at the crown, 4 ft. 8 in. at the midway point, and 6 ft. 3 in. at the abutment. The rise at the crown was 20°, the setting temperature being 88 degrees. At the midway point, the rise varied from 27 to 40°, according to the distance from the surface, the setting temperature varying from 82 to 95 degrees. At the abutment the rise varied from 30 to 50°, according to the distance from the surface, and the setting temperatures varied from 95 to 115 degrees. In every case, the maximum temperature was reached within 4 days.

In the Walnut Street Bridge, in Des Moines, Iowa, to which reference was made by the writer in his paper on "Concrete Bridges: Some Important Features in Their Design",* the average setting temperature, as given by nine thermometers set at various points in the arch, was 94.5°, the maximum being 108 and the minimum 84 degrees. The average rise was 17.1°, with a maximum of 32 and a minimum of 6 degrees. The reason for the small rise is that the initial temperature of the concrete is given as very high, being 77.4°, with a maximum of 80 and a minimum of 76 degrees. The average time in which the maximum temperature was reached after placing, was 33.6 hours, with a maximum of 75 and a minimum of 10 hours. The writer, therefore, is convinced that if the concrete had been richer in cement, the maximum temperature would have been reached in much less than 30 days.

Conclusions (2) and (3) do not seem to be borne out by Fig. 7, but seem to be justified by Fig. 8, and, as the record in Fig. 8 was taken 2 months after placing and that of Fig. 7 immediately after placing, probably the former is more reliable. It would be interesting

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 695.

Mr. W. M. Smith. to know the reason for this discrepancy. The writer thinks it may be due to the following cause: The thermometers were placed in August, when the sun's effect was much greater than at the end of September, when the record given in Fig. 8 was taken. As Thermometer No. 7 was near the down-stream face, which is sloping and has a western exposure, the sun was almost perpendicular to the concrete and at a time of day when its effect was much more powerful. This may account for the fact that Thermometer No. 7 shows a greater range at this time than Thermometer No. 9, although the former is 1 ft. farther from the face.

Mr. Wiley. A. J. WILEY,* M. AM. SOC. C. E. (by letter).—The authors deserve the thanks of the Profession for undertaking and carrying out in such detail experiments which every engineer who has designed a masonry dam has felt the need of, temperature changes in mass concrete being usually neglected. Apparatus for determining the temperature in the interior of a mass of masonry has been available for only a few years, but, now that it has been perfected, it is to be hoped that no important masonry dam will be constructed without the installation of such simple and inexpensive apparatus.

In most masonry dams, the only evidence of interior temperature is the cracks which occur as the masonry cools. It appears to have been established by the experiments of the authors, as well as by those made with similar apparatus on the Boonton Dam, that Portland cement masonry—whether Cyclopean concrete containing 50% of large rubble rock, as in the Boonton Dam, or ordinary concrete to which about the same proportion of large cobbles has been added, as in the Arrowrock Dam—reaches a setting temperature of about 100° Fahr., if placed during the summer.

As the rise in temperature is due to the chemical action which causes the setting, the heating and hardening of the masonry must be simultaneous, so that the mass assumes its final form when its temperature is highest. Consequently, there is no compressive stress in the masonry to be relieved by the subsequent cooling, the entire range in temperature, between 100° and the lowest temperature to which the masonry cools, being converted into tensile stress.

The heat conductivity of the masonry does not permit its interior to acquire the temperature of the air, and the interior temperature varies in some proportion to its distance from the nearest face. The Boonton experiments indicate that the range of interior temperature varied directly as the range of exterior temperature and inversely as three times the cube root of the distance from the nearest face. The Arrowrock experiments do not seem to confirm this, as the only seasonal result given by the authors shows a range of 32° in the masonry 3.5 ft. from the nearest face, with an atmospheric range of 75°, whereas

* Boise, Idaho.

Mr. Wiley.

the Boonton experiments would indicate a range of 16° for these conditions.

The cracks resulting from the contraction of the masonry are a great detriment to the appearance of a dam, and no amount of care in the construction will prevent them. As they are the result of contraction from the expanded condition of the mass due to heat generated in the setting of the cement, masonry containing a large proportion of cement is more likely to crack than the leaner mixtures.

The usual rise in temperature of a large masonry mass above the temperature of the materials entering into the construction is about 35°, and if the seasonal variation 3.5 ft. from the surface of the masonry is 32°, it would seem that, if the masonry could be placed at an average temperature of 17.5° below the average seasonal temperature, there would be no tension in it at distances more than 3.5 ft. from the surface. This emphasizes the importance of laying as great a percentage of the masonry as possible during the winter, and it has been the writer's experience that it is possible to prevent temperature cracks, to a very great extent, if not entirely, by this method.

The Granite Springs Dam, designed and built under the writer's supervision, for the City of Cheyenne, Wyo., is a masonry structure of ordinary gravity section. It is 10 ft. long at the base and 450 ft. at the crest, with an extreme height of 96 ft., and is built on a curve with a radius of 300 ft. It is of uncoursed rubble, with granite rocks, up to 3 cu. yd. in volume, embedded in 1:4 Alpha Portland cement mortar. It contains 0.613 bbl. of cement per cu. yd. of masonry, and 35% of its mass is mortar and 65% granite rock.

The first masonry was laid on April 20th, 1903, which, at this altitude of 7 000 ft., is about as early as it is practicable to do such work without protection. Work was suspended on November 20th, on account of the temperature, and resumed on April 11th, 1904. The dam was completed on June 21st, 1904. During the season of 1903, about 80% of the dam was built, the central portion being entirely completed, leaving a gap at each abutment. In the spring of 1904, the gap at the east end was first completed, and then that at the west end, the last masonry in this section being laid on June 21st.

In the following winter, a single temperature crack—the only one ever discovered in the dam—opened up about 100 ft. from the west end, near the middle of the last section laid. It was barely perceptible, but extended, in a vertical radial plane, entirely around the dam, which, at this point, is 30 ft. high and 17 ft. wide at the base. In general, the crack followed the mortar joints, but at one point a large facing stone was cracked about half way across its face. As far as known no leakage ever occurred through the crack, and, though the writer has made no examination since the winter of 1904-05, he is

Mr. Wiley. informed by the present city engineer that neither the original nor any other crack is noticeable at the present time.

It has been claimed that the use of racking or stepping in the masonry tends to form cracks. At each end of the first completed central portion of the dam, the masonry was racked or stepped off, at a slope of about 45° for a height of 50 ft., but no cracks ever appeared anywhere near these joints.

The seasonal range of temperature at this dam is about 130°, and the writer is of the opinion that the freedom from cracks is due to the fact that the two ends were built after the central portion had become contracted by cooling during the winter; he also believes that, had the gap at the west end been filled at the same time as the one at the east end, while the temperature was still low, the small crack in this section would also have been avoided.

The Salmon River Dam, which was designed and built under the writer's supervision, is 30 miles south of Twin Falls, Idaho. It is of Cyclopean concrete, 215 ft. high, 15 ft. thick at the top, 116 ft. thick at the base, 480 ft. long on top, and 320 ft. long at the base. It is curved in plan, with a radius of 225 ft. at the up-stream face. It contains 21.3% of lava stone up to 5 tons in weight, and 78.7% of concrete formed of Portland cement, crushed lava, and natural sand in the proportions of 1 part of cement to 7⅓ parts of concrete, or 0.71 bbl. of cement per cu. yd. of masonry.

The placing of concrete was started on June 1st, 1909, and completed on December 1st, 1910. During this time, work was carried on continuously for 20 hours per day, except for periods aggregating about 4 weeks during the winter of 1909-10, when it was suspended on account of the extreme cold. The seasonal range of temperature was 135°, from 3° below zero to about 130° above, in the sun. Provision was made for heating the concrete before placing, and concreting was carried on in all but the very coldest weather.

Early in the spring of 1910, a fine crack appeared on the up-stream face of the dam about 150 ft. from its east end. It did not appear on the down-stream face, and has now entirely disappeared. With the exception of a few hair cracks on the thin parapet walls, this is the only crack that has ever shown in this dam.

The construction was carried on in strictly horizontal layers, but two openings, each about 10 ft. wide, on radial lines, were kept in it for a height of about 80 ft. above the base, to carry the river during construction. These openings had smooth vertical sides, with occasional V-shaped notches, 1 ft. on a side, to act as cut-offs. No other precaution was taken to prevent seepage when filling the openings with concrete, and no seepage has ever taken place in these vertical joints, nor have any cracks ever appeared near them. In striking contrast to this, it was found that, whenever operations on a

horizontal surface were suspended long enough to permit thorough setting of the concrete, no precautions were sufficient to prevent seepage along the joint thus formed. Mr. Wiley.

The dam of the Swan Falls Power Plant is on the Snake River, about 30 miles south of Boise, Idaho. It is 460 ft. long, with an extreme height of 40 ft. and an average height of 25 ft. It is a plain concrete wall, with a uniform thickness of 5 ft., supported by buttresses of the same thickness at intervals of 22 ft. between centers. In plan, it consists of two straight sections, that on the north being 160 ft. and that on the south 360 ft. in length. The concrete is a mixture of Portland cement and natural river gravel, the lower part mixed in the proportion of 3 sacks of cement to 1 cu. yd. of concrete, the upper 10 ft. having 2½ sacks to 1 cu. yd.

The first concrete was laid about August 1st, 1900, and the dam was completed about the middle of October, with the exception of an opening for drainage purposes in the middle of the longer section. This opening was 6 ft. wide, with vertical sides, and extended to the base of the dam. It was closed by filling it with concrete in December, 1900, at which time it is thought that the setting temperature had not entirely disappeared, but there has been only one crack in the dam. This is about ⅛ in. wide, and occurs at the angle point in the dam, leaving one straight section of 160 ft. and one of 300 ft. entirely free from cracks.

It is the writer's opinion that cracks in masonry dams could be prevented entirely by leaving vertical openings at intervals of from 100 to 200 ft. in each season's work, to be filled at the earliest practicable date in the beginning of the next season, the temperature of the masonry being then at the minimum.

He believes that the same result could be achieved more conveniently in some cases by providing vertical joints at intervals of about 100 ft. in the length of the dam. These joints should have an expansion filler at the up-stream and down-stream faces, and a system of vertical openings, a few inches in diameter, converging at the top of the dam, through which cement grout under heavy pressure could be forced after the dam had lost its setting heat and at the period of lowest temperature for the interior masonry.

J. Waldo Smith,* M. Am. Soc. C. E.—This matter of tests to determine temperatures in dams, so that an idea of the stresses may be arrived at, has occupied the attention of engineers for about 13 or 14 years. For this purpose, probably the Boonton Dam was the first structure in which thermophones were placed and observations made. These results have been reported to the Society, and some tentative conclusions have been drawn. The theory, perhaps, is some- Mr. J. W. Smith.

* New York City.

Mr. J. W. Smith. what like that which a geologist draws; but it certainly is a good thing to have these observations and facts in the records of the Society; and, perhaps, at some later time, from the accumulation of these facts, it may be possible to draw some conclusions which will be of value.

Of course, what is desired is to know what stresses are set up in the masonry, and whether such stresses act in the direction of the load or opposed to it, and that is something which is most difficult to ascertain. No one has yet suggested a scheme by which those stresses could be determined, other than by deducing them from the temperatures taken at the Boonton Dam. The thermophones used then were not perfected, as they are now, and after a time they deteriorated to such an extent that accurate observations could not be made; and from the experience thus gained, the makers are now able to make instruments which seem to be trustworthy and maintain their reliability for a long time.

That very great strains are set up in masonry is known from observations of copings and various other constructions in which large slabs are scaled off due to the expansion and contraction resulting from the temperature changes, and also from the cracks which are formed, not only transversely, but also longitudinally, if the dam is more than 150 or 200 ft. thick.

At the Boonton Dam there were a number of sections in which the temperature cracks showed longitudinally as well as at the transverse sections provided for expansion. At the Kensico Dam, Mr. Seabury has distinguished these very carefully, and the results will be interesting when he gets all the data together, and particularly after the observations have been followed out for a year or two longer.

In the design of the fixed-arch, reinforced concrete highway bridges of the New York City Board of Water Supply, the stresses due to temperature changes were considered, the information desired having been the maximum temperature ranges throughout the arch. In order to obtain such temperatures for structures of this type, thermophones were placed in one of the bridges, but, as in the case of the Kensico Dam, the information is not yet available.

This scheme of taking temperatures reminds one of the pastime of physicians in taking blood pressure—in measuring the force of the heart in some way. The wise ones say, "we do not know whether or not this will be of any use to us, but we are diligently taking these observations from day to day, and hope that some time in the future we will be able to evolve some theory or some facts that will be exceedingly useful".

CHARLES H. PAUL,* M. AM. SOC. C. E., and A. B. MAYHEW,† ASSOC. M. AM. SOC. C. E. (by letter).—Since the preparation of the paper, nearly a year ago, additional data have been accumulated, none of which tends to contradict any of the original conclusions. Messrs. Paul and Mayhew.

The upper half of Plate XVII gives the thermometer record for the whole year, 1914, amplifying the partial record for that year shown in the lower half of Plate XVI; and the lower part of Plate XVII gives the record for 1915, as far as available. This latter brings out several interesting points.

Compare the records of Thermometers Nos. 7 and 8, the former being 2 ft. from an exposed face and the latter 10 ft. from the same face, and at the same elevation and cross-section. The record of Thermometer No. 7 shows the effect of the daily variations of outside temperature, and Thermometer No. 8, 8 ft. farther in from the face, gives a smooth curve, and shows very clearly the comparatively slight effect of daily variations of outside temperature. With a seasonal variation in mean daily temperatures of 73°, the seasonal variation of Thermometer No. 8, 10 ft. in from the face, was only 12 degrees. A careful comparison of these two records, together with that of the air temperatures, makes an interesting study. The hump in the curve of Thermometer No. 7, during the latter part of August, where the recorded temperature exceeds the mean daily temperature, is due to the fact that the maximum daily temperatures during that period were in the neighborhood of from 95 to 100°, and the warm part of the day continued for more than its share of the time, so that a mean of hourly temperature readings would give a result considerably higher than the mean daily temperature as computed ordinarily.

Comparison of the records of Thermometers Nos. 7 and 9 is interesting in showing the performance of two thermometers at the same elevation, each close to an exposed face. Thermometer No. 7 is 2 ft. from the down-stream face (west exposure) and Thermometer No. 9 is 1 ft. from the up-stream face (east exposure). The water in the reservoir was above the elevation of Thermometer No. 9 from May 11th until August 24th, 1915.

The record of Thermometer No. 5, as compared with that of Thermometer No. 9, shows the effect of the water in the reservoir on the temperature of the concrete close to the up-stream face. The water was above the elevation of Thermometer No. 5 after February 26th, and the temperature of the water at that elevation was between 32° and about 52° during the remainder of the period shown on Plate XVII. At the elevation of Thermometer No. 9 (covered from May 11th to August 24th), the water was considerably warmer, especially

* Arrowrock, Idaho.

† Dayton, Ohio.

Messrs. Paul and Mayhew.

after the early part of August, when the surface was not far above that elevation.

The records for Thermometer No. 4 now extend over a period of nearly 2½ years. It was covered at first to a depth of 3½ ft. and left thus for several months during the summer of 1913. Late in October, 1913, concreting over this thermometer was resumed, and the effect of this was felt until about February 1st, 1914, when the temperature registered was 71 degrees. Since that time the temperature has been falling very slowly and steadily. It has reached 58° at the date of this writing (September 1st, 1915), and an examination of the curve during that period shows how very little the temperature at that distance from an exposed face is affected by the seasonal variations of air temperatures. There is just a slight gradual change in direction of the curve responding with a lag of several months to the extremes of seasonal temperatures.

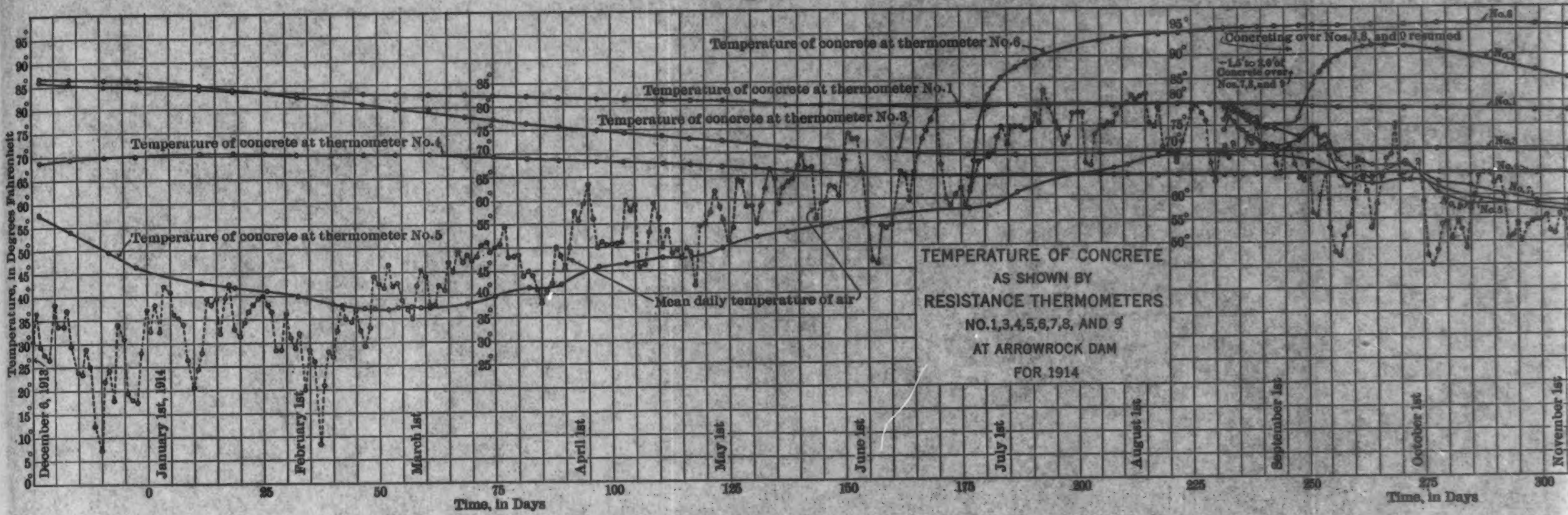
The curves for Thermometers Nos. 1 and 4 are almost parallel (since February, 1914), as would be expected. A comparison here shows that the original temperature of the concrete makes its effect felt for a very long time. Ultimately, we would expect these two thermometers to register temperatures not far apart, with No. 4 slightly higher than No. 1. Assuming that No. 1 will finally register about 40°, which is approximately the mean temperature of the water at the bottom of the reservoir, and that it will continue to drop at the same rate as it has for the last 12 months, it will have taken 5 years after placing for it to reach its fixed temperature.

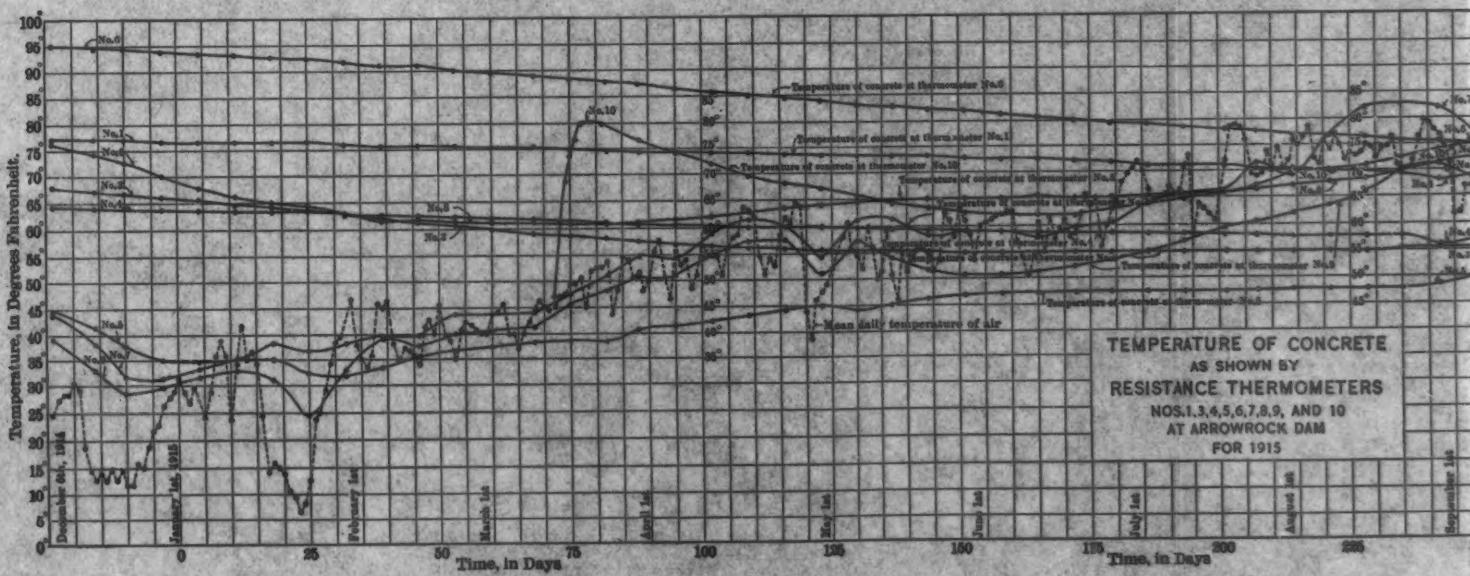
Thermometer No. 10 was placed on March 12th, 1915, in fresh concrete, and covered to a depth of 2 ft.; within 24 hours it was covered to a total depth of about 6 ft. Its record shows the results that would be expected after a study of the others.

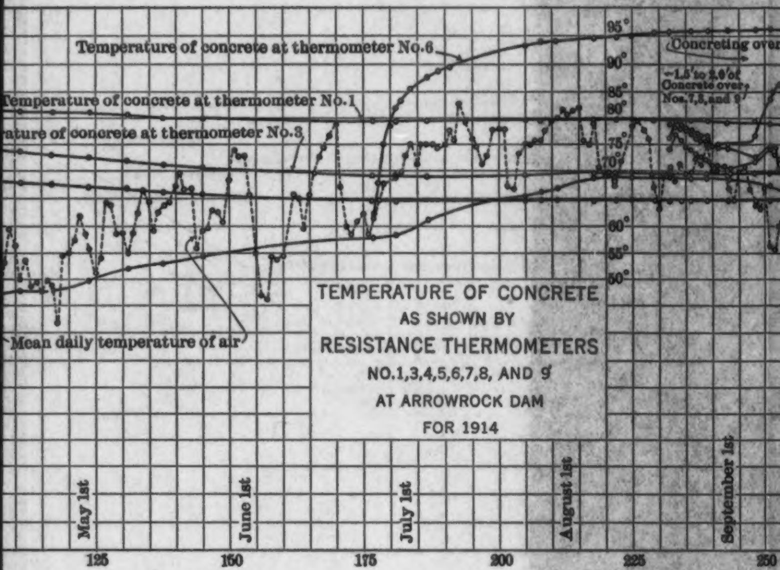
With records of nearly 2½ years for some of the thermometers, and after having the reservoir filled within 28 ft. of the top of the dam during the season of 1915, all the thermometers (with the exception of No. 2) continue to give readings that indicate perfect condition of the apparatus. This is very gratifying, in view of the difficulties experienced heretofore in obtaining satisfactory results for any length of time with similar apparatus.

For the benefit of those who may wish to go further into the effect of water in the reservoir, Table 1 gives the record of the elevations of water surface at various dates during the season of 1915.

The writers are pleased to know that investigations similar to those undertaken at Arrowrock are being carried on at Kensico. It is gratifying, though not surprising, that in all essential points the results of these two independent researches have led to similar conclusions.


TEMPERATURE OF CONCRETE
AS SHOWN BY
RESISTANCE THERMOMETERS
NO.1,3,4,5,6,7,8, AND 9
AT ARROWROCK DAM
FOR 1914
Temperature of concrete at thermometer No.6
Temperature of concrete at thermometer No.1
Temperature of concrete at thermometer No.3
Temperature of concrete at thermometer No.4
Temperature of concrete at thermometer No.5
Mean daily temperature of air
Temperature, in Degrees Fahrenheit
Time, in Days
December 6, 1913
January 1st, 1914
February 1st
March 1st
April 1st
May 1st
June 1st
July 1st
August 1st
September 1st
October 1st
November 1st


TEMPERATURE OF CONCRETE
AS SHOWN BY
RESISTANCE THERMOMETERS
NOS.1,3,4,5,6,7,8,9, AND 10
AT ARROWROCK DAM
FOR 1915
Temperature of concrete at thermometer No.6
Temperature of concrete at thermometer No.1
Temperature of concrete at thermometer No.10
Mean daily temperature of air
Temperature, in Degrees Fahrenheit.
Time, in Days
January 1st, 1915
February 1st
March 1st
April 1st
May 1st
June 1st
July 1st
August 1st
September 1st


TEMPERATURE OF CONCRETE
AS SHOWN BY
RESISTANCE THERMOMETERS
NO.1,3,4,5,6,7,8, AND 9
AT ARROWROCK DAM
FOR 1914
Temperature of concrete at thermometer No.6
Temperature of concrete at thermometer No.1
ature of concrete at thermometer No.3
Mean daily temperature of air
95°
90°
85°
80°
75°
70°
60°
55°
50°
May 1st
June 1st
July 1st
August 1st
September 1st
125
150
175
200
225
250

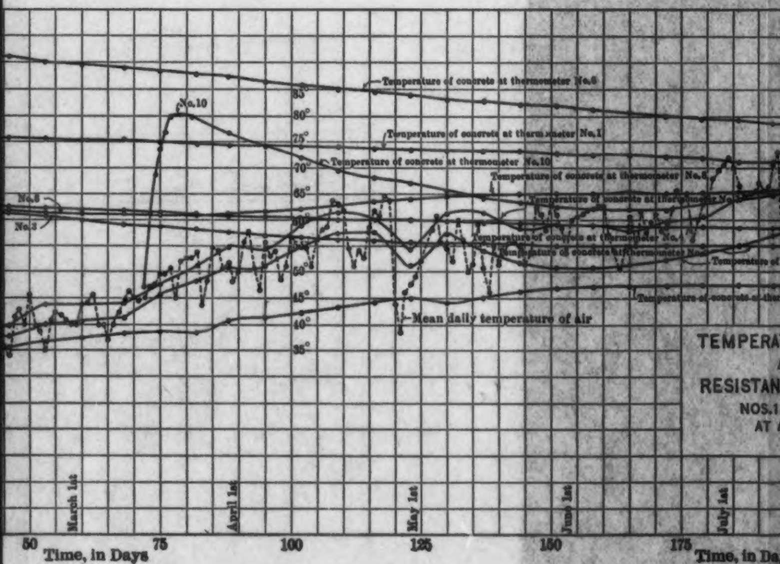
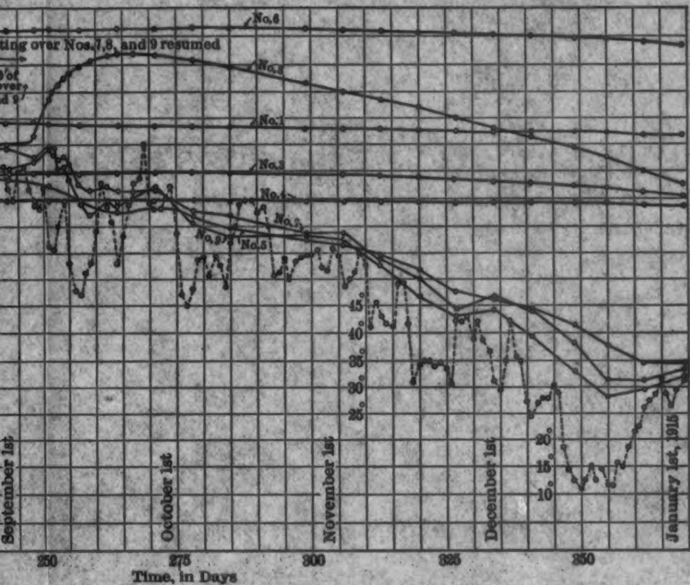
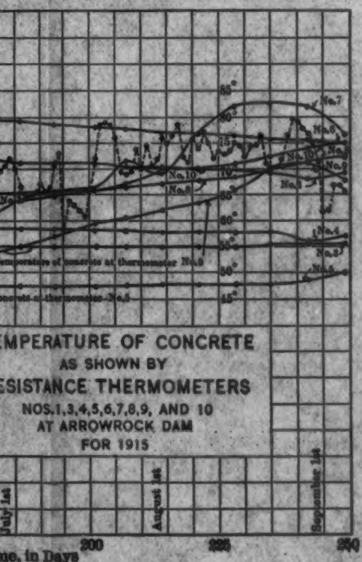

TEMPERA
RESISTAN
NOS.1
AT
No.10
Temperature of concrete at thermometer No.6
Temperature of concrete at thermometer No.1
Temperature of concrete at thermometer No.10
Mean daily temperature of air
No.8
No.3
85°
80°
75°
70°
65°
50°
45°
40°
35°
March 1st
April 1st
May 1st
June 1st
July 1st
50
75
100
125
150
175
Time, in Days
Time, in Da

PLATE XVII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXIX, No. 1349.
PAUL AND MAYHEW ON
TEMPERATURE CHANGES IN
MASS CONCRETE.





Messrs. Paul and Mayhew.

TABLE 1.—ELEVATIONS OF WATER SURFACE IN ARROWROCK RESERVOIR.

Date.	ELEV. WATER SURFACE. Rising.	Date.	ELEV. WATER SURFACE. Falling.
February 15th	2 972	June 12th	3 183
March 1st	3 020	July 1st	3 171
" 15th	3 033	" 15th	3 155
April 1st	3 049	August 1st	3 136
" 15th	3 069	" 15th	3 118
May 1st	3 085	September 1st	3 086
" 15th	3 119		
June 1st	3 173		
" 12th	3 183		

Mr. Seabury misunderstands the intentions of the writers when he states: "At the Arrowrock Dam, an effort seems to have been made to reduce to a minimum the effect of the setting cement, * * *." The only object the writers had in mind was to obtain a record of temperature changes in mass concrete under average conditions. Some of the thermometers were covered a few inches and some as much as 3½ ft.—most of them from about 1½ to 2 ft.—when placed. Some were placed close to an exposed face, and some well back in the mass. The writers believe that this gives records very close to the average for the class of work represented by large dams, where it is not usual to place a layer of concrete more than from 2 to 4 ft. in thickness, at any one place, during one shift. Maximum possible temperatures are undoubtedly of great interest, but the writers believe that average temperatures —or perhaps average maxima for any considerable volume, like a day's work—are more valuable; for the practical use of these data is in their application to the design of dams or other large structures, where good-sized blocks may be considered as units, and where the use of maximum possible temperatures, obtained under conditions not corresponding to actual practice, might lead to unwarrantable results. It has been the writers' experience that concrete in large dams is usually placed in about 2-ft. layers, and that it is not usual to place more than two consecutive layers over any considerable area during one shift.

Keeping in mind the fact that the writers are considering average results for masses of several hundred yards spread in layers, and that Mr. Seabury, if understood correctly, is thinking of maximum possible temperatures in masses smaller in volume, or at least in superficial area, it is not difficult to bring into accord the opinions on any essential points. For instance, referring to Mr. Seabury's comments on Conclusion (1), radiation always does take place, to a large extent, under ordinary conditions of placing mass concrete, and very rich mixes are not used ordinarily in work of that class.

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Sand-cement concrete sets more slowly than straight Portland-cement concrete, and probably, therefore, the maximum temperature of a mass of sand-cement concrete placed under ordinary conditions would be lower than that of Portland concrete similarly placed.

Mr. Seabury's record of the experiments with Alsen and Atlas cement concretes placed under similar conditions is very interesting. No mention is made as to which is the quicker-setting cement, but the writers would guess that the Atlas was a little the quicker, and that, under ordinary conditions of placing, it would show a slightly higher maximum than the Alsen.

Mr. Wilson Fitch Smith's observation *A* corresponds very closely with the results obtained at Arrowrock. His observation *B* suggests this comment, that during the time the temperature is increasing on account of chemical action the concrete is losing moisture, which tends to lessen the effect of the increasing temperature on its change of volume. The writers believe, however, that Mr. Smith has suggested a more or less serious objection to the too rapid placing of concrete in confined masses with comparatively small superficial areas. In general, as he states, his conclusions are similar to those of the writers.

Without being able to give results of experiments to support their opinions, the writers would answer Mr. Immediato's questions as follows:

1.—A quick-setting cement will probably give higher temperatures, both intermediate and final, than a slower-setting cement, assuming that the concrete is spread in layers over comparatively large areas, and that all conditions are similar.

2.—The effect of free lime is not known.

3.—Generally speaking, the denser concrete would give higher values.

4.—Excess of water would probably tend to decrease the maximum temperatures.

Regarding Mr. Williams' first comment, it is impracticable, of course, to check the thermometers after they are once placed; but by comparing them with one another, it is possible to judge as to their accuracy. The records obtained at Arrowrock have been very satisfactory in that respect, and the troubles mentioned by Mr. Seabury have not been encountered there—always excepting Thermometer No. 2, which went bad soon after it was placed. It might be said, concerning this thermometer, that apparently it is slowly recovering, although as yet no records, except those shown on Plate XVI, are being used.

Mr. Walter M. Smith gives some very interesting data in regard to temperature changes in arches. As these sections are all comparatively thin, and the concrete is comparatively rich, the conditions are hardly

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comparable with those at Arrowrock, and different results, therefore, would be expected.

As to the quantity of Portland cement in the Arrowrock concrete compared with the ordinary 1:3:6 mix, the point should not be overlooked that in sand-cement the Portland content is very finely ground, and therefore more of it is active than in commercial Portland cement. It has been shown at the Arrowrock laboratory that about 30% of commercial Portland cement is retained on a 280-mesh sieve, and that this 30% is practically inactive when used as cement. Probably some that just passes this sieve is also inactive, but no apparatus was at hand to determine that; however, it is not unreasonable to assume that at least 35% of commercial Portland cement is inactive. This residue, when reground to pass a 200-mesh sieve, is more active than ordinary Portland cement. Of the sand-cement, as manufactured at Arrowrock, only 15% is held on a 280-mesh sieve, and of that residue practically all is blending material, so that almost all the Portland in sand-cement passes the 280-mesh sieve, and is finer ground and more active than the corresponding 70% of commercial Portland. The difference in active cement content between the 1:2½:5 sand-cement concrete used at Arrowrock, and ordinary 1:3:6 Portland cement concrete, therefore, is very small. The 2¾ parts of cobbles used at Arrowrock corresponds to the plum rock used commonly in mass work. It is true, however, that the paper deals with comparatively lean mixtures, as are common for mass work, and Mr. Smith's comment on that point is well founded.

Conclusions (2) and (3) were based on the results shown by Fig. 8, where, it will be noted, temperatures of air and concrete were taken every 2 hours for a 48-hour period, and after the effect of chemical action was fairly well overcome. Fig. 7, covering a period immediately after the placing of the thermometers, would not be expected to give reliable data on that particular point.

Mr. Wiley shows clearly the advantage of cold-weather work, which is beyond question. Unfortunately, however, it is not always possible to avoid concreting during the warmer months, and, in any case, the writers believe that contraction joints are desirable.

It is Mr. Wiley's discussion that opens up the question of contraction joints, which properly goes with the subject matter of this paper. Fig. 9 shows the design and spacing of contraction joints in the Arrowrock Dam.

By taking advantage of the inspection galleries, especially the one at Elevation 3 090.5, it has been possible to measure the movement of fifteen of these joints (spaced 50 ft. apart at that elevation), not only at the up-stream face of the dam, but also at the up-stream and down-stream walls of the inspection gallery, 16 ft. and 22 ft., respectively, in

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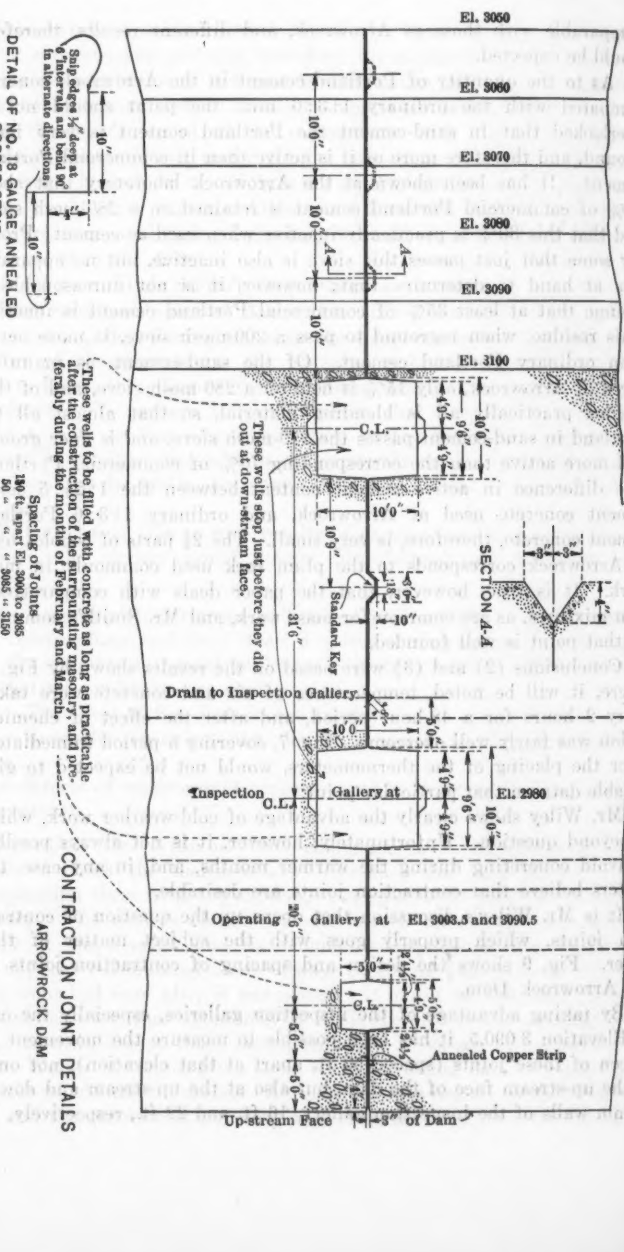

FIG. 9.

from the up-stream face of the dam. The average maximum opening of these fifteen joints was as follows: Messrs. Paul and Mayhew.

At up-stream face....................	0.0708 in.
16 ft. in from up-stream face...........	0.0444 "
22 " " " " "	0.0324 "

The average minimum opening of these fifteen joints, subsequent to the maximum opening, was:

At up-stream face......................	0.0064 in.
16 ft. in from up-stream face...........	0.0300 "
22 " " " " "	0.0240 "

These records are for one season only, and undoubtedly the heat in the concrete, due to setting, was still being felt, but they are given for what they are worth. Later records along this line will be of interest and value. It is believed, however, that these records, meager as they are, indicate the value of contraction joints in structures of this kind.

The writers would add to the "Conclusions" given in the paper, the following:

(6) (Superseding No. 6 in paper.) The seasonal variation in the temperature of concrete 10 ft. from an exposed face is about 12°, when the seasonal variations of the mean daily temperature of the air is about 72 degrees.

(7) The seasonal variation in the temperature of concrete 20 ft. from an exposed face is very little, and after the effect of "setting heat" has once been overcome, the change in temperature of concrete at that distance from an exposed face is so slight as to be negligible, under ordinary conditions.

(8) The effect of the "setting heat" in concrete 20 ft. or more from an exposed face is felt for several years after the concrete is placed. Concrete near the center of mass of a large dam probably retains some of this "setting heat" for 5 years or more after placing.

(9) Contraction joints are desirable in all large concrete structures, whether or not they are to be exposed to wide variations of outside temperature, except possibly when all construction may be carried on during cold weather, and when the variation of outside temperatures, after completion, will be slight.

(10) Changes in volume due to setting, hardening, and seasoning of concrete, are more important, ordinarily, excepting very close to exposed faces, than changes in volume due to the influence of daily or seasonal variations in outside temperatures.

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Paper No. 1350

THE PUMPING PLANT OF THE MORENCI WATER COMPANY*

By W. L. Du Moulin, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Alexander Potter, Lindsay Duncan, H. Hawgood, and W. L. Du Moulin.

Synopsis.

The aim of this paper is to describe the pumping plant of the Morenci Water Company, and to relate operating experiences and difficulties encountered in its construction.

The plant is about 6 miles southwest of Morenci, Ariz., on the Eagle River. It furnishes all the water used for domestic purposes by the inhabitants of the Town of Morenci and also by the two copper mining companies operating there.

The town is an unincorporated mining camp in Greenlee County, in the middle of the rough, mountainous region of southeastern Arizona. Copper mining is the sole industry, and is responsible for the camp's existence. The two mining companies are The Detroit Copper Mining Company, owned by Phelps, Dodge and Company, operating a 1 300-ton mill and a 350-ton smelter; and The Arizona Copper Company, Limited, controlled by Edinburgh capital, operating a 3 000-ton mill. The smelter of this company is at Clifton, Ariz. The population of Morenci is about 6 000, 20% being American and 80% Mexican.

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* Presented at the meeting of June 2d, 1915.

The physical character of the country surrounding Morenci, and in the direction of the pumping plant particularly, is exceedingly rough, rocky, and mountainous, and practically barren—in fact, almost a desert.

There is no water obtainable near the town. A small quantity was formerly pumped by the mining companies from the San Francisco River, some 9 miles distant, stored in small tanks, and carried around camp on the backs of burros, mules, and Mexicans, but it was very dirty and unwholesome. The Morenci Water Company was formed in October, 1898, to pump water for domestic purposes from the Eagle River. This water is distributed by gravity to the houses around camp through pipe lines from storage tanks, and is very good and wholesome. In 1908, a larger plant was needed to supply the increased demand for domestic purposes. At the same time, the mining companies decided to abandon pumping for their own use the bad water from the San Francisco River, and made arrangements with the water company for the better supply from the Eagle River. To provide the quantities required by the mining companies and for domestic purposes, the water company built the present plant, with its machinery of special design. This enlarged plant, with its efficient equipment, has thus replaced a number of small plants with inefficient equipment and high pumping costs, and has pumped the larger quantity of water at much less cost.

This is perhaps the only pumping plant in the United States furnishing water to a community for domestic purposes, where water is delivered in one lift through about 5 miles of pipe lines against a static head of 1 525 ft. It consists of three pumping engines, two being triple-expansion, condensing, high-duty engines, and one cross-compound, condensing, high-duty engine, with the necessary boilers, etc. The total capacity of the plant is 4 500 000 gal. per 24 hours.

The whole plant, together with the distributing system, etc., represents a valuation of $675 000.

Structures.

The pumping station contains a main engine-room, a smaller engine-room, a boiler-room, and an addition for miscellaneous reserve equipment, and covers an area of 176 ft. 0 in. by 78 ft. 6 in. The building proper is of substantial steel construction, consisting of a

structural steel frame on a concrete foundation, with corrugated-steel sides and roof. The addition is of corrugated-steel construction, but with wood frame. In the main engine-room, there is a 10-ton, Maris Brothers, hand, traveling crane of 45-ft. span. The floor of this room is of reinforced concrete, but that of the smaller engine-room is of wood. The basement floors of these and the boiler-room are on the same level and are of concrete. The building is well lighted.

Fig. 5 is a diagrammatic plan of the plant, showing the principal features.

A steel bridge, with a span of 188 ft. and a width of 9 ft., carries the three 10-in. water mains over the Eagle River.

There are three steel fuel-oil storage tanks at the plant. A 100 000-gal. tank, 30 ft. high, and a 40 000-gal. tank, 18 ft. high, are on the same side of the river as the plant, and a 200 000-gal. tank, 30 ft. high, is on the opposite side.

The settling basins are along the cliff above the plant. Their location relative to the plant is shown on Fig. 16. They are described in more detail later.

On the side of the river opposite the plant, there are six cottages for the employees of the company. They are of substantial wooden construction, each having electric lights, running water, etc.

Equipment.

In the main engine-room there are two Nordberg, horizontal, four-cylinder, triple-expansion, condensing, poppet-valve, direct-acting, crank and fly-wheel type, pumping engines. The smaller engine-room contains one Nordberg, horizontal, cross-compound, condensing, direct-acting, crank and fly-wheel type, pumping engine, having poppet-valves on the high-pressure and Corliss valves on the low-pressure cylinders. The pump end of this engine extends into the main engine-room in order to be within reach of the 10-ton traveling crane. In both the triple-expansion and the cross-compound pumping engines, the cranks are set at an angle of 90 degrees.

Triple-Expansion Pumping Engines.—The arrangement of the triple-expansion, poppet-valve, pumping engines is similar to that of an engine at the Gneisenau shaft of the Harpener Bergwerks Gesellschaft, in Germany. The pump end in each engine is behind the steam cylinders, of which there are four. There are no connections

FIG. 1.—MORENCI, ARIZ. THE TWO WATER STORAGE TANKS ARE ON THE HILL.

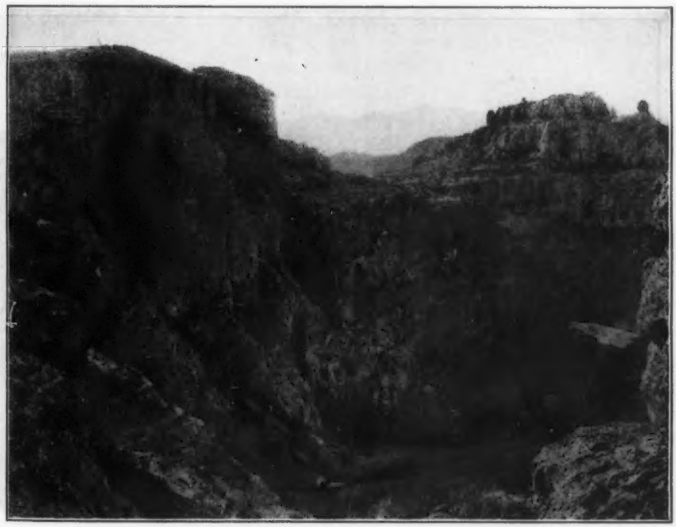


FIG. 2.—COUNTRY IN THE VICINITY OF MORENCI.

[illegible]

FIG. 12.—COUNTRY IN THE VICINITY OF MURRAY.

FIG. 3.—PUMPING PLANT, MORENCI WATER COMPANY. SETTLING BASINS IN THE DISTANCE.

FIG. 4.—PUMPING PLANT, MORENCI WATER COMPANY.

Fig. 5.

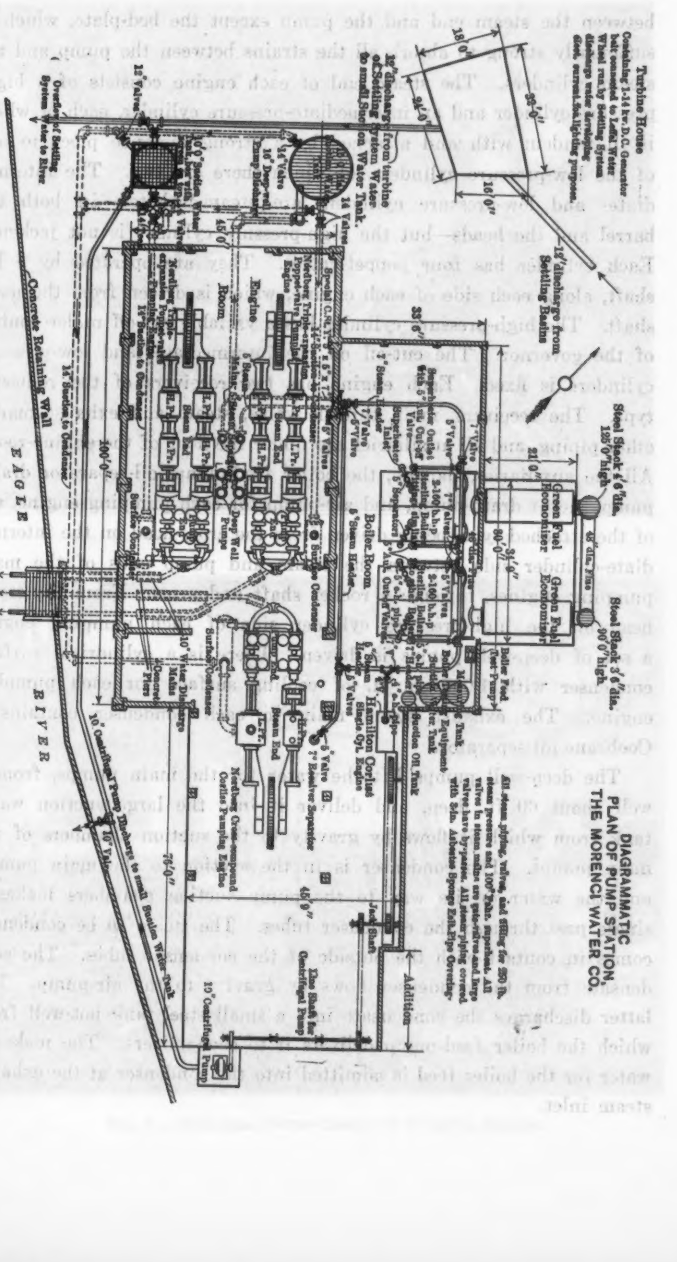

PLAN OF DIAGRAMMATIC PUMP STATION, THE MORENCI WATER CO.
Turbine House
Settling Basins
Steel Stack 5'6" dia. 125'0" high
Steel Stack 3'6" dia. 90'0" high
Green Fuel Economizer
Boiler Room
Engine Room
Deep Well Pump
Concrete Retaining Wall
EAGLE RIVER
Steel Bridge
Addition

FIG. 5.

between the steam end and the pump except the bed-plate, which is sufficiently strong to absorb all the strains between the pump and the steam cylinders. The steam end of each engine consists of a high-pressure cylinder and an intermediate-pressure cylinder, each of which is in tandem with and attached by a strong distance piece to one of the low-pressure cylinders, of which there are two. The intermediate- and low-pressure cylinders are steam-jacketed—in both the barrel and the heads—but the high-pressure cylinder is not jacketed. Each cylinder has four poppet-valves. They are operated by a lay shaft, along each side of each engine, which is driven from the main shaft. The high-pressure cylinder has a variable cut-off under control of the governor. The cut-off on the intermediate- and low-pressure cylinders is fixed. Each engine has two receivers of the re-heater type. The receivers, main throttle valve, steam and exhaust mains, other piping, and all auxiliaries are under the floor of the engine-room. All the auxiliaries, namely, the boiler feed-pump, oil-separator drain-pump, jacket drain-pump, and air-pump for each pumping engine, are of the attached type, being driven from the cross-head on the intermediate-cylinder side between the steam and pump ends of the main pumping engines, through a rocker shaft and arms. From the cross-head on the high-pressure cylinder side of each pumping engine a set of deep-well pumps is driven. There is a cylindrical surface condenser with 1 000 sq. ft. of cooling surface for each pumping engine. The exhaust steam main to each condenser contains a Cochrane oil separator.

The deep-well pumps lift the water for the main pumps, from a well about 60 ft. deep, and deliver it into the large suction water tank from which it flows by gravity to the suction chambers of the main pumps. The condenser is in the suction to the main pumps, and the water on its way to the pump suction chambers makes a single pass through the condenser tubes. The steam to be condensed comes in contact with the outside of the condenser tubes. The condensate from the condenser flows by gravity to the air-pump. The latter discharges the condensate into a small steel tank hot-well from which the boiler feed-pump delivers it to economizers. The make-up water for the boiler feed is admitted into the condenser at the exhaust steam inlet.

FIG. 6.—TWO NORDBERG, TRIPLE-EXPANSION PUMPING ENGINES.

FIG. 7.—NORDBERG CROSS-COMPOUND PUMPING ENGINE.

The pump end is of cast iron, and was designed especially strong to work against the heavy total hydraulic head of about 1 700 ft. The pumps are of the horizontal, duplex, double-acting, outside, center-packed plunger type. The plungers are of chilled cast iron. The pump-valves are of the double concentric ring type, of cast steel, faced with fiber washers. The seats are of cast iron, strongly ribbed, and have a central stud of brass passing through and guiding the valve. Each valve seat has an open area of 65 sq. in. The lift of the valve can be controlled by the adjustment of a spring. The total weight of a pump-valve and seat is about 425 lb. Each pump contains eight valves—four suction and four discharge—one valve to a chamber, as shown on Fig. 8.

Each triple-expansion pumping engine is designed to pump "1 000 U. S. standard gal. of water per min., against a total head of 750 lb., at a piston speed of 300 ft. per min.; and, when running condensing, is guaranteed to do 175 000 000 ft-lb. of work for every 1 000 lb. of steam evaporated from hot-well temperature with a steam pressure at the throttle of 165 lb., gauge, and the steam superheated 100° Fahr."

Table 1 gives the results of a duty test on one of these pumping engines by a responsible testing engineer. The quantity of steam to be charged to the engine was obtained by weighing the steam condensed by the condenser and in the jackets and re-heaters. The leading dimensions of this engine are given in Table 2.

TABLE 1.—Test of Triple-Expansion Pumping Engines.

Duration of test	8 hours.
Average steam pressure at throttle	170 lb., gauge.
Average steam pressure at first receiver	34 "
Average steam pressure at second receiver	0 "
Average superheat at throttle	142° Fahr.
Average water pressure	720 lb.
All water pumped was also lifted out of a well a distance of	35 ft.
Duty per 1 000 lb. of steam	181 197 500 ft-lb.
Steam per indicated horse-power per hour	10.23 lb.
Mechanical efficiency	93 per cent.
The pumping engine ran slightly over capacity.	

Cross-Compound Pumping Engine.—The horizontal, cross-compound, pumping engine was designed to pump "1 000 U. S. standard gal. of water per min., against a total head of 750 lb., at a piston speed of 300 ft. per min.; and, when running condensing, is guaranteed to do 160 000 000 ft-lb. of work for every 1 000 lb. of steam evaporated

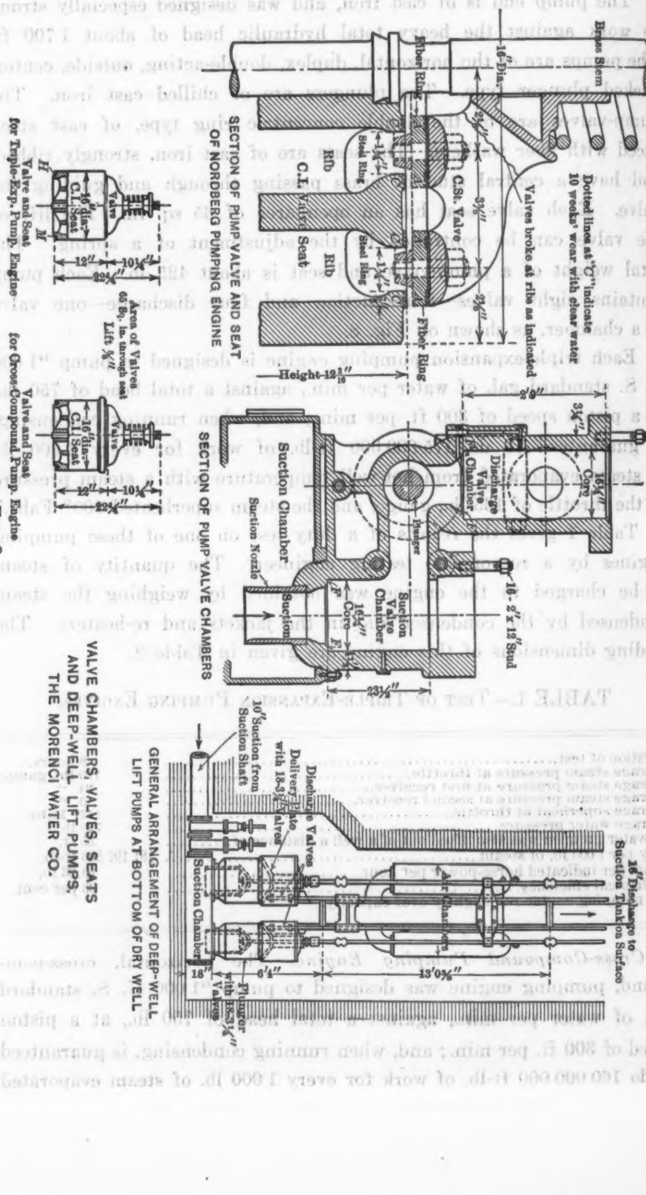


FIG. 8.

VALVE CHAMBERS, VALVES, SEATS AND DEEP-WELL LIFT PUMPS, THE MORENCI WATER CO.

TABLE 2.—PRINCIPAL DIMENSIONS OF TRIPLE-EXPANSION PUMPING ENGINES.

High-pressure steam cylinder	20-in. diameter, 42-in. stroke.
Intermediate-pressure steam cylinders	38-in. diameter, 42-in. stroke.
Low-pressure steam cylinder	40-in. diameter, 42-in. stroke.
Plungers (two)	6¾-in. diameter, 42-in. stroke.
Plunger rods (two)	2$^{13}/_{16}$-in. diameter.
Piston rods, head end, high-pressure and intermediate-pressure cylinders	3⅞-in. diameter.
Piston rods, head end, low-pressure cylinders and crank end, high-pressure and intermediate-pressure cylinders	4$^{11}/_{16}$-in. diameter.
Piston rods, crank end, low-pressure cylinders	4⅞-in. diameter.
Fly-wheel (weight = 27 000 lb.)	16-ft. diameter, 10 in. wide.
Main bearings	13¾ by 25 in.
Crank pins	6¾-in. diameter, 15 in. long.
Cross-head pins	5-in. diameter, 8⅛ in. long.
Cross-head shoes	12 by 20 in.
Foundation dimensions	14 ft. 0 in. by 57 ft. 1⅛ in.

from hot-well temperature with a steam pressure of 155 lb., gauge, at the throttle, and the steam superheated 100° Fahr."

Table 3 gives the results of a duty test on this engine, running slightly over capacity, by a responsible testing engineer. The quantity of steam used was determined as in the case of the test on one of the triples.

TABLE 3.—TEST OF CROSS-COMPOUND PUMPING ENGINE.

Duration of test	10 hours.
Average steam pressure at throttle	155.7 lb., gauge.
Average water pressure	725.5 lb.
Average superheat in steam at throttle	107.8° Fahr.
Mechanical efficiency	96 per cent.
Duty per 1 000 lb. of steam	171 620 000 ft-lb.
Steam per indicated horse-power per hour	11.1 lb.

The pump end and pump valves of this engine, in the main, are identical with those of the triple-expansion engines. There are two steam cylinders—a high-pressure and a low-pressure. The high-pressure cylinder has four poppet-valves, and these are operated by a lay shaft, along the side of the engine, driven from the main shaft. The cut-off is variable, being under the control of the governor. The low-pressure cylinder has Corliss valves, with the cut-off controlled by hand. The high- and low-pressure cylinders are both jacketed; the former in the heads only, the latter in both barrel and heads. The auxiliaries are of the attached type, being driven from the cross-head between the pump and steam ends, as in the triple-expansion engines. This pump, however, has no deep-well lift-pumps attached,

as it pumps principally settling system water for the concentrators. The condenser and air-pump combined are similar to those of the triple-expansion engines.

The leading dimensions of this pumping engine are given in Table 4.

TABLE 4.—PRINCIPAL DIMENSIONS OF CROSS-COMPOUND PUMPING ENGINE.

High-pressure steam cylinder	23-in. diameter, 42-in. stroke.
Low-pressure steam cylinder	52-in. diameter, 42-in. stroke.
Plungers (two)	6¾-in. diameter, 42-in. stroke.
Plunger rods (two)	2¹³⁄₁₆-in. diameter.
Main bearings	13¾ by 25 in.
Crank pins	6¾-in. diameter, 15 in. long.
Cross-head pins	5-in. diameter, 8½ in. long.
Cross-head shoes	12 by 20 in.
Piston rods, crank ends	4⅞-in. diameter.
Piston rods, head end	3⅞-in. diameter.
Fly-wheel (weight = 27 000 lb.)	16-ft. diameter, 10 in. wide.
Foundation dimensions	18 ft. 0 in. by 47 ft. 10 in.

The triple-expansion and cross-compound pumping engines are equipped with Richardson force feed oil pumps for steam cylinder lubrication; and for the lubrication of the bearings, etc., there is a complete gravity oiling system, with filters, oil pumps, tanks, etc. Wherever practicable, grease cups are used.

OPERATING EXPERIENCES.

Usually, it is necessary to run two pumping engines, and at such rates as the demand for water requires, for the water storage at Morenci is very limited. Consequently, the rate of running is not regular and not as uniform as is usually the case in the average water-works pumping plant. The average duty developed by these pumps, under the usual operating conditions, during a period of one month, varies from 155 000 000 to 162 500 000 ft-lb. per 1 000 lb. of steam chargeable to the engines; that is, the duty developed comes within from 10 to 5% of that which would probably be developed under similar test conditions. Experience has shown that the plant performance is not as good when two triples are running as when a triple and the cross-compound are running. Under the usual conditions, the cross-compound produces better results back at the boiler end than a triple, although, under test conditions, the results favor the triple.

As is perhaps usual, to a certain extent, in machinery of new and special design, unsatisfactory features developed in these pumping engines which led to a great deal of trouble and expense. The pump valves caused the most serious trouble. The original pump valves were of cast steel, and the seat was of cast iron, weighing approximately 400 lb. These valves were not suitable for the conditions and proved extremely unsatisfactory, as they generally began to leak within a few days after being placed in the pump. Under favorable conditions, with clear water, the wear on valve and seat, in the course of 10 weeks, was as indicated by the dotted lines at *A* and *C* on Fig. 8. Consequently, both valve and seat had to be refaced at considerable expense once every 10 weeks and sometimes oftener. In addition to this heavy repair expense, the life of the valve seat was naturally very short. The wear on the seat was usually uneven, and caused the ribs of the valve to crack, as indicated on Fig. 8. The total maintenance cost of this item of valves and seats was extremely high. Furthermore, with the use of these valves, the quantity of water actually pumped was from 72 to 85% of the plunger displacement. Finally, a cast-steel valve, faced with a fiber washer, was tried. This is shown by the light dotted lines on Fig. 8. These washers are ½-in. thick, are held in place by ½-in. studs and a steel ring, are able to withstand the great pressure to which their bearing surface is subjected, and are giving satisfaction. The wear has been transferred from the 400-lb. cast-iron seat to the fiber washers, which are easily replaced. These washers leak a trifle when a valve is first placed in the pump, but soon pound themselves to the seat, after which they never leak. The life of a set of washers is about 4 months.

Another unsatisfactory feature developed in connection with the pump-valve seats. Fig. 8 shows a section through part of a pump valve and seat, and a section of the pump chambers. When a valve and seat are placed in a chamber, the bottom of the valve seat, *H*, rests on the bottom of the chamber, *F*. A round gasket is placed at *M*. The result is a joint approaching a ground joint. A spindle fits into the top of the brass stem and is screwed down on it after the cover has been placed on the chamber. This is the only means, outside of the action of the water pressure, and its own weight, of forcing and holding the valve seat down on the chamber bottom in the case of the triples, and is not capable of insuring positively such a result.

It does not accomplish its purpose with absolute certainty in the very place where such a feature is of vital importance. As a consequence, with the original cast-steel valves, a leaky condition frequently existed at the bottom of the valve seats, and there was no means of detecting it. There was constantly the danger of the bottoms of the valve chambers becoming seriously cut.

In the case of the cross-compound pumping engine, which was erected later, the design was changed by putting a beveled shoulder on each valve seat, on which four lugs, in the side of each valve chamber, acting as wedges, are forced. This design has proved more satisfactory, but not entirely so. With the present pump valves faced with fiber, a leak at the bottom of the seat can usually be detected, and so the danger of seriously cutting the chamber bottoms is not so great.

With the present valves faced with fiber, and changes in the throat bushings of the plunger stuffing-boxes, the average quantity of water pumped during 1913 amounted to 95.02% and during 1914, 95.3% of the plunger displacement.

The triple-expansion engines were originally equipped with air chambers, but as it was impossible to retain sufficient air in them to be of benefit, the pumps have been running without them. At this great water pressure, the air was either forced through the pores in the cast iron, of which the air chambers are made, or absorbed very rapidly by the water. There was no economical method of replenishing it rapidly enough. At the same time, up to date, there have been no difficulties in running without air chambers—due perhaps to the irregularity of the 10-in. lines conveying the water to Morenci. Without doubt, the small air pockets along the lines are sufficiently numerous to absorb the usual shocks due to the pulsations of the strokes of the plungers.

The tests of the pumping engines mentioned previously were conducted after the changes had been made in the pump valves and plunger stuffing-boxes, and other improvements had been effected.

Another very unsatisfactory feature in connection with the pumping engines is the vacuum-producing equipment with which they were furnished. The vacuum obtained is comparatively poor. The equipment for each engine consists of a cylindrical surface condenser and an air-pump of the attached type. The surface condensers for both

the triples and the cross-compound have each 1 000 sq. ft. of cooling surface, which, according to the usual practice, should be ample to produce sufficient cooling to obtain a very good vacuum. Each condenser is 42 in. in diameter, and contains 270 1½-in. outside diameter 16 B. w. g., seamless drawn brass tubes, 9 ft. 6 in. long. The exhaust steam enters the condenser at one end at the top and comes in contact with the outside of the tubes. A baffle-plate extends along the upper portion of the condenser so that the steam makes one pass. The condenser is slightly inclined, and the condensate leaves it at the bottom, directly opposite the exhaust steam inlet, and flows by gravity to the air-pump. There are no baffles other than the one mentioned.

The type of air-pump furnished for each condenser is the single-acting, vertical, bucket, combined wet and dry type, 17 in. in diameter, with 16-in. stroke. In the case of the triples, the suction of the air-pump has no foot- or check-valve. The air-pump for the cross-compound has a foot-valve in the suction and a separate air connection with a valve from the surface condenser, which leads into the pump between the discharge-valve check and the bucket.

The altitude at the plant is about 3 600 ft. and the corresponding barometer reading about 26.5 in. Although, at times, there is a better vacuum, it generally averages 18.5 in. for the triples, the corresponding back pressure of which is 3.93 lb. This was the average vacuum from the day the pumps were first started. The temperature of the condensate, water vapor, and air mixture in the suction of the air-pump is 127° Fahr. On the cross-compound, the average vacuum is about 20.5 in.; the corresponding back pressure of which is 2.95 lb.; and the temperature of the condensate, water vapor, and air mixture in the suction of the air-pump is 135° Fahr. Consequently, in the case of the triples, the total of 3.93 lb. in the condenser is made up of 2.04 lb. steam pressure and 1.89 lb. air pressure; and, in the case of the cross-compound, the total of 2.95 lb. in the condenser is made up of 2.53 lb. steam pressure and 0.42 lb. air pressure. From these figures, it is quite apparent that there is insufficient cooling in both cases. The figures indicate, furthermore, that the air-pump displacement for the triples is not large enough to extract the mixture of condensate, air, and uncondensed vapor as completely as is necessary to maintain a good vacuum. The ratio of the air-pump displacement to the low-pressure cylinder displacement is 1:25 for the cross-com-

pound and 1: 30 for the triples. For this type of attached air-pump, practice has established ratios of 1: 9 and 1: 16, which give continued good results. The quantity of circulating or cooling water is more than 100 lb. per pound of steam to be condensed, which is abundant. The rise in temperature of this water through the condenser is only about 10°, and the difference between the temperature of the condensate mixture and this water after it has passed through the condenser is between 50 and 70° Fahr. The insufficient cooling, therefore, is not due to a lack of sufficient cooling water to abstract the latent heat in the steam. In view of the fact that there is an abundance of water and that the extent of cooling surface is adequate, it would seem that the insufficient cooling is due to the fact that the design of the condenser is not the proper one to insure that all the surface is fully effective.

Although the question of vacuum is not mentioned in connection with the duty guaranties, the fact of the matter is that the design of the entire vacuum-producing equipment is not in line with the careful, high-class design of the rest of the steam end of the engines.

In general, considering the severity of the service, these engines have been running very smoothly, and the results have been very satisfactory.

The engines complete, with all auxiliaries, including condensers and air-pumps, were built especially for this service by the Nordberg Manufacturing Company, of Milwaukee, Wis.

Boiler-Room Equipment.

The boiler-room equipment consists of four Sterling, Class A, No. 11, water-tube boilers, set in two batteries, each battery containing two boilers. Each boiler is rated at 160 boiler h.p., is constructed for a working steam pressure of 160 lb., is equipped with a Hammel oil-burning furnace, adapted to local conditions, and one Hammel improved oil burner. A fuel-oil pump delivers the oil to the burners where steam is used to atomize it. Each boiler is also equipped with the Foster attached type of superheater, guaranteed to superheat 6 000 lb. of steam per hour at 160 lb. steam pressure, 125° Fahr., with a drop in pressure through the superheater of 1½ lb. or less. On each boiler, in the tubing carrying the saturated steam, between the boiler nozzle and superheater inlet, there is a steam separator with its drain

connected to a trap. This separator extracts any undue quantity of moisture, and thus assists the superheater by delivering to it a comparatively dry saturated steam. It also protects the superheater in case a boiler foams and large quantities of water are carried over with the steam. The drain at the superheater inlet is also connected to a trap.

In the rear of each battery of boilers there is a Green fuel economizer, containing 120 tubes, with 1 440 sq. ft. of heating surface, completely encased in brick. The two economizers are protected by a building of corrugated steel, with a wood frame.

The boiler-room is the most important part of a power-plant, especially in such an isolated place as Morenci, and in a country where fuel is costly, as large losses may occur there. Too much care cannot be exercised to determine what changes to effect and the proper routine to establish in order to obtain the best results under the operating conditions.

A great deal of attention has been given to the boiler settings. After the brickwork had been placed in first-class shape and given several coats of paint, the outside of the setting, especially in the region of the furnace and combustion chamber, was covered with a coating of asbestos several inches thick. All points of air leakage were stopped up, and the ends of the steam drums and mud drum, the flues in rear of the boilers, economizer connections, and all small piping around the boilers were covered with 2 in. of asbestos. The entire settings and flues, etc., were then covered with several layers of thick, unbleached muslin and given several coats of thick paint. On the brickwork on the top of the boilers, a layer of asbestos several inches thick was placed, and this was covered with a layer of bricks and cement. All the main steam piping has been covered with J-M Asbesto-Sponge felted pipe covering, 3 in. thick. A daily inspection of all settings in use is made, and all points of air leakage are stopped up and painted. In this way, the possibility of air infiltration through brickwork and at connections is reduced to a minimum; and the settings are maintained in first-class condition continually. The proper quantity of baffling inside the boiler settings has been determined by experiment, and the furnaces have been built in a manner which brings the best results. When burning fuel oil, the construction of the furnace, together with the manner of bringing the air to the burners, is important.

Miscellaneous Reserve Equipment.

The principal reserve equipment consists of a Hamilton-Corliss valve, single-cylinder, 12 by 36-in. steam engine, which runs a 10-in. Krogh horizontal centrifugal pump through a jack-shaft and a line of shafting, as indicated on Fig. 5. This engine, in conjunction with the centrifugal pump, is held as a reserve unit to lift water from the river into the small suction water tank, in case of an accident to the settling system, or to one of the deep-well pumps. This engine also furnishes power for any other purposes.

Fuel Oil System.

At Morenci, the fuel oil is emptied directly from railroad tank cars into two steel receiving tanks. A Gould triplex plunger pump, with a capacity of 40 gal. per min., geared to a 10-h.p. Westinghouse, mill type, induction motor, for which electric power is purchased from one of the mining companies, pumps the oil through a 4-in. pipe to the top of a hill about 300 ft. high, from which it flows by gravity through about 26 000 ft. of 6-in. pipe to the plant. As this pipe line runs over very rough country, there are many inverted siphons in its length. It contains about 20 000 gal. of oil, the quantity varying somewhat with the season of the year. Whenever oil is pumped over at Morenci, it flows into a 200 000-gal. tank which is about 100 ft. above the other two tanks. The latter are then filled from the 200 000-gal. tank as occasion requires, but never from the tank out of which oil is being used. Oil is drawn into the two oil-measuring tanks, from either the 100 000-gal. or the 40 000-gal. storage tank. From the measuring tanks, the oil flows by gravity to the fuel-oil pump which heats it and delivers it to the burners under a pressure of 60 lb. This enables the firemen to control his fire better than would be the case if he depended on gravity flow, and the pump uses no more steam than would be required to heat the oil sufficiently to give a free flow to the burners under gravity feed. The total fuel oil regularly on hand at the plant amounts to 310 000 gal.

The fuel oil is from California, gravity 18° Baumé, at a temperature of 62° Fahr. Pumping at the rate of 36 gal. per min., through 1 110 ft. of 4-in. pipe, up the hill at Morenci, temperature of oil 70° Fahr., the pressure at the pump is 160 lb.; the static pressure is 140 lb. During warm weather, it requires about 3 hours to send

over to the plant 6 500 gal. of oil. During cold weather, however, it requires about a day. As a general thing, no difficulty is experienced during ordinary cold weather; but when it is extremely cold, it is necessary to build fires along the line at regular distances, to heat the oil, so as to enable it to be pumped.

Water Transmission Mains Between Plant and Morenci, and Water Storage Tanks at Morenci.

There are three 10-in. lines conveying the water from the plant to Morenci. Fig. 9 is a plan and profile of one of the lines showing also the wagon road to the plant. The lines are within view from this road for their entire distance. Each pump discharges through two 8-in. bends and a Y-connection into its own 10-in. line. There are no cross-over connections between the lines.

At Morenci, the lines discharge into two steel storage tanks, each 53 ft. 6 in. in diameter and 30 ft. high, and each having a capacity of approximately 500 000 gal. One is designated the Town Tank, and into it only well water is pumped. The town distributing system receives its water from the Town Tank, and in this way the people are supplied with well water for domestic purposes. The mining companies take their boiler feed water from the town system. The other tank is designated the Concentrator Tank, and into it is pumped the settling system water. It also receives the overflow from the Town Tank. The fluctuations in water level occur in the Concentrator Tank and, consequently, it is provided with a high- and low-water alarm attachment. From the Concentrator Tank, water is delivered to the concentrators of both mining companies through 10- and 12-in. pipe lines, and these two lines are entirely separate from the town distributing system. All the water pumped into the storage tanks is measured by Venturi meters as it leaves them.

The 10-in. pipe lines from the plant to Morenci are laid on the surface of the ground, and conform to the topography of the country over which they run. Each line is about 25 650 ft. long, and contains 3 000 ft. (near the plant), weighing 54.74 lb. per ft.; 18 000 ft., weighing 40 lb. per ft.; and 4 650 ft. (near Morenci), weighing 35 lb. per ft. The pipe near the plant is extra strong, ½ in. thick. All pipe is of wrought steel, threaded for 2½ in. with standard threads, and screwed together with recessed line couplings. Near the plant, each line has two

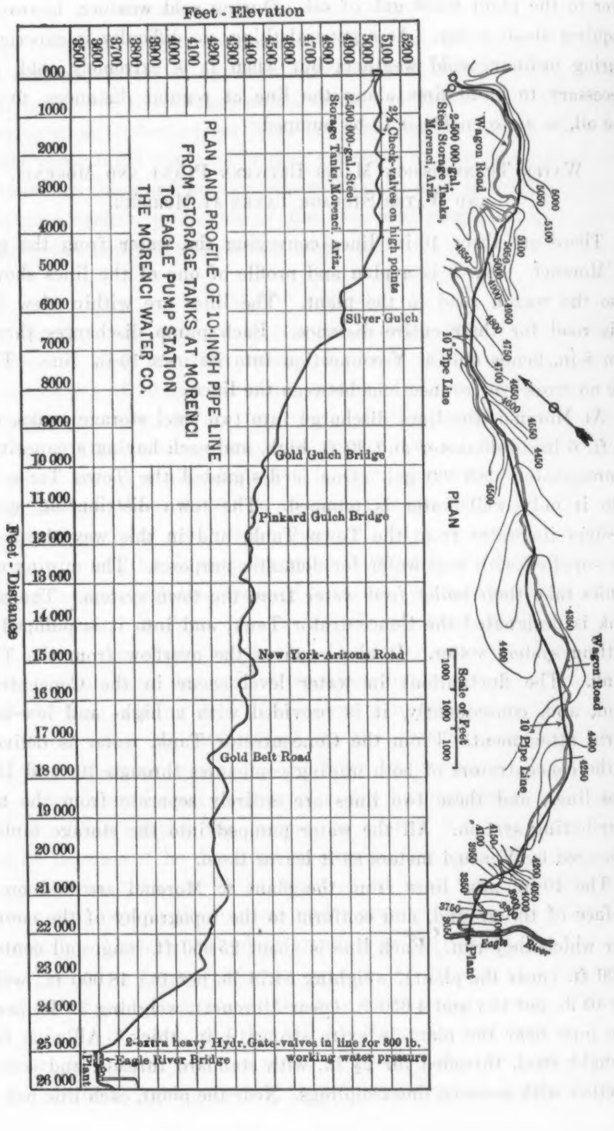

Feet - Elevation
PLAN AND PROFILE OF 10-INCH PIPE LINE
FROM STORAGE TANKS AT MORENCI
TO EAGLE PUMP STATION
THE MORENCI WATER CO.
2½ Check-valves on high points
2-500 000-gal. Steel Storage Tanks, Morenci, Ariz.
2-500 000-gal.
Steel Storage Tanks,
Morenci, Ariz.
Wagon Road
Silver Gulch
10 Pipe Line
Gold Gulch Bridge
Pinkard Gulch Bridge
PLAN
New York-Arizona Road
Scale of Feet
1000 1000 1000
Gold Belt Road
2-extra heavy Hydr. Gate-valves in line for 800 lb.
working water pressure
Eagle River Bridge
Plant
Eagle River
Feet - Distance

Fig. 9.

Fig. 10.—Discharge Pipes from Pumps in Pumping Station, Showing Method of Anchoring.

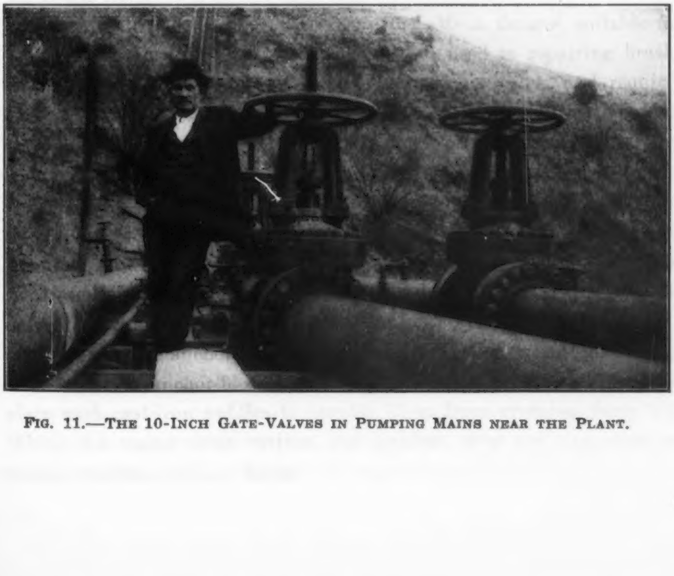


Fig. 11.—The 10-Inch Gate-Valves in Pumping Mains near the Plant.

10-in., Crane, flanged, extra heavy, hydraulic, ferro-steel, outside screw and yoke, wedge gate-valves, with hard metal seats and 1½-in. by-pass, designed for a cold-water working pressure of 800 lb.

Originally, each line contained also at this point a 10-in., Crane, extra heavy, hydraulic, ferro-steel, swing check-valve, designed for 800-lb. cold-water working pressure. Breaks in the lines between the pumping plant and the check-valves occurred with great frequency. It was noticed that the swinging disk of each check-valve pounded against its seat in unison with the pulsations of the pumps and set up a severe water-hammer. The check-valves were removed, and since then there have been no breaks in that portion of the lines.

There are no expansion joints in the pipe lines. The contraction and expansion in lines as exposed as these is very great, but is compensated by the large number of inverted siphons which they contain.

There are no fittings in the lines other than those mentioned. As the mains were being laid, all bends were made by heating the pipe in place and bending it to the desired deflection. Very satisfactory bends, which are neither flattened nor buckled, were made in this manner, and thus the lines were made to conform to the country with as few strains as possible in them. At the plant, the mains are carried over the Eagle River on a steel bridge and a wooden trestle.

Crane, extra heavy, hydraulic, ferro-steel, 10-in. flanges, suitable for a cold-water working pressure of 800 lb., are used in repairing breaks in the pipes. Each is 21 in. in diameter, 2¾ in. thick, and requires twenty 1¼-in. bolts. However, in the portion of the lines near the plant, these flanges were found to be unsatisfactory, as they cracked and gave way repeatedly. It was found necessary, therefore, to use a special flange, forged from a single billet of wrought steel, without weld, and finished to the same dimensions as the Crane ferro-steel flanges. Experience has shown that plain-faced flanges are best suited to the conditions, and that a ring gasket of vulcanized fiber, $\frac{1}{32}$ in. thick, is the only kind which gives complete satisfaction.

The lines are anchored only near the plant, where there are several large concrete anchor-blocks, well buried in the ground, and held in place with cast-iron saddles to prevent them from creeping down hill. Where the mains cross ravines and gulches, they are supported on wooden trestles, without floors.

In the plant building, near the discharge of the pumps, it was found necessary to anchor each line thoroughly to piers and to support and brace all bends rigidly. At a point some 4 000 ft. from the water storage tanks at Morenci, the lines have very nearly the same elevation as the tops of the tanks. Here they form a siphon the vertical height of which between the extremities and the highest point is much more than 27 ft., which is about the theoretical height of a column of water that the air pressure at this altitude will sustain. Consequently, between pump pulsations, there is a break in the water column in each line at this point, and a partial vacuum forms, resulting in a very severe metallic water-hammer. Rapid corrosion of the pipe also takes place here. Thirty 2½-in., horizontal-swing, check-valves were placed on each line at this place, with a distance of 75 ft. between each. These admit air when there is a tendency for a vacuum to form, and confine the air when the pulsations force the water forward, thereby creating a cushion for any water-hammer. The air churns up with the water, resulting in an enlarged volume of a mixture of air and water. These valves have practically eliminated the water-hammer, and also, it is believed, the excessive corrosion of the pipes at this place.

The static head between the discharge of the pumps and the tops of the tanks at Morenci is 1 525 ft. When pumping through a 10-in. line, at a rate that represents 1 000 gal. of actual water per minute, the total head amounts to 1 700 ft. This makes the friction head about 175 ft. in 25 650 ft. of smooth, screwed, wrought-steel pipe; or 6.82 ft. of friction head per 1 000 ft. of length.

Water-Supply Sources.

There are two sources of supply. Water for domestic and boiler feed purposes is obtained from a well. A settling system furnishes the large quantities used by the concentrators of the mining companies. Fig. 16 is a plan of this system, showing the tunnels, flumes, ditches, and settling basins, and their relative location with respect to the plant.

Wells.—The original well supplied all the water formerly pumped into Morenci, the maximum being about 300 gal. per min. This well consists of an unlined shaft, about 8 ft. square and 62 ft. deep, sunk in a conglomerate formation. A tunnel about 70 ft. long, also in conglomerate, connects this shaft with the gravel forming the river bed, at the edge of the conglomerate formation. This gravel bed is the

Fig. 12.—Bridge and Wooden Trestle Carrying Pumping Mains across the River.

Fig. 13.—A View of the Pipe Lines.

source of the well water. With the building of the present plant, a shaft, 9 ft. 6 in. by 7 ft. 6 in. and 60 ft. 6 in. deep, containing the suction heads of the deep-well pumps, was sunk at a point some 80 ft. from the building. This shaft is in gravel, and is timbered for the first 22 ft. The remainder is in conglomerate. Two 10-in. suction pipe lines connect this shaft to the dry well, which is between the two Nordberg engines. The dry well is sunk to a depth of 62 ft. and is lined with concrete 3 ft. thick, with a clear 6 by 9-ft. opening. At the bottom of the dry well, there is an 8 by 12-ft. station containing the four cylinders of the deep-well pumps. Their construction is shown on Fig. 8. Each deep-well pump has two cylinders, and is driven from a triple-expansion pumping engine, as already explained. Each of the 10-in. suction lines from the suction shaft connects directly to a deep-well pump, which lifts sufficient water for one pumping engine. There are no check- or foot-valves in the suction lines of these deep-well pumps. A tunnel through conglomerate connects the suction shaft with the original well.

It required but a short time, pumping at a rate of from 1 200 to 1 300 gal. per min., which was the quantity desired at the completion of the present plant, to demonstrate that the yield of the original well was about 700 gal. per min. Holes were dug along the river bank, and from there it was noticed that the water-table of the gravel forming the river bed had become lowered. It was evident that the ground-water which accumulates in the gravel of the river bed was not gathering rapidly enough, through the natural percolation of the surface water, to maintain a sufficient supply in the well. Continued pumping seemed to show, also, that the permanent supply was a portion of a definite underground flow, the main body of which seemed to be at a greater depth, according to all indications. To maintain the supply of the well at the desired quantity, holes were dug in the gravel bank, on the same side of the stream as the plant, about 700 ft. up stream, and water from the river was turned into them. Each hole was about 20 ft. square and 14 ft. deep. There are no difficulties in obtaining sufficient well water in this way until the rainy season comes on. At that time, the floods bring down muddy water and silt, and the latter seals the river bottom and the sides and bottoms of the sump holes, thereby reducing the percolation of water to such an extent that the well supply drops to about 700 or 800 gal. per min. Until last year, the additional

water required at that season was pumped into the small suction-water tank directly from the river by a centrifugal pump. From this tank, it was delivered to the concentrator-water tank in Morenci by a separate pumping engine and pipe line.

The pumping of muddy water during the rainy season very materially increased the pumping costs, seriously interfered with the operation of the plant, and presented many difficulties. The water contained so much silt and gritty matter that the condenser tubes were clogged to such an extent that the pumps could not be operated. It was necessary occasionally to clean the condensers several times in 24 hours. The grit in the water cut the pump valves and seats so much that they had to be changed every 3 or 4 days, and sometimes in less than 48 hours; also small sticks and foreign matter caused the valves to hang up, with the resultant injury to which pumps and pipe lines are subjected when under a total head of 1 700 ft.

At all times of the year, the well supplies sufficient water for domestic purposes and for boiler feed. It is delivered into the Town Tank by a separate pumping engine and pipe line. To obtain a sufficiently large quantity of satisfactory water for the increasing concentrator requirements, a settling system was installed.

Settling System.

The settling system consists of a series of settling basins, blasted out of the conglomerate along the cliff, near and some 65 ft. above the plant. The first basin is formed by a concrete dam, about 14 ft. high, 5 ft. wide at the base, 2 ft. wide at the top, and 72 ft. long, built across a gulch at the outlet of Tunnel *D,* and has a water surface area of 3 600 sq. ft. This dam is well reinforced, resting on solid rock, and has an additional reinforcement of two abutments. A second settling basin, some 260 ft. long, 21.7 ft. average width, and containing a total water surface area of 5 600 sq. ft., was blasted out of solid conglomerate along the cliff. A concrete wall, 2 ft. wide and 3 or 4 ft. high, was built on the outer side of this basin, to make it of uniform elevation. In addition, there is the settling effect of the ditch, 300 ft. long and about 5 ft. wide, giving a water surface area of about 1 500 sq. ft. The total water surface area of the main settling basins is about 10 700 sq. ft. In addition to these main basins, there is an auxiliary settling basin, of concrete, 50 by 66 ft., which is beyond the reach of floods,

Fig. 14.—A View on the Pipe Lines.

Fig. 15.—One of the Flumes.

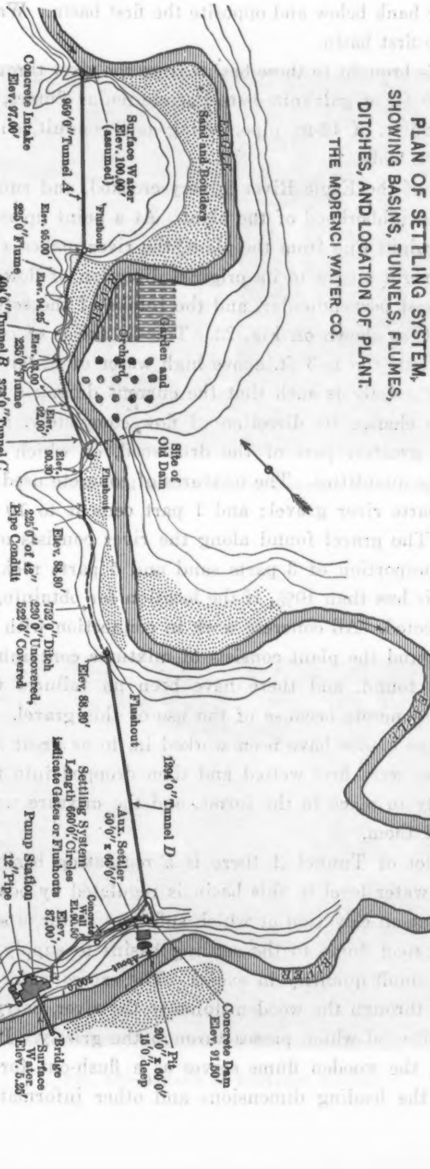

PLAN OF SETTLING SYSTEM,
SHOWING BASINS, TUNNELS, FLUMES,
DITCHES, AND LOCATION OF PLANT.
THE MORENCI WATER CO.
EAGLE
RIVER
Sand and Boulders
Garden and
Orchard
Site of
Old Dam
Surface Water
Elev. 100.00′
(assumed)
Flushout
959′0″ Tunnel A
Concrete Intake
Elev. 97.00′
Elev. 95.00′
Elev. 94.25′
225′0″ Flume
Elev. 93.00′
235′0″ Flume
Elev. 92.00′
404′0″ Tunnel B
373′0″ Tunnel C
Elev. 90.30
225′0″ of 42″
Pipe Conduit
Elev. 88.80′
752′0″ Ditch
230′0″ Uncovered,
522′0″ Covered
Flushout
Elev. 88.80′
1294′0″ Tunnel D
Aux. Settler
50′0″ x 66′0″
Settling System
Length 660′0″. Circles
indicate Gates or Flushouts
Concrete
Wall
El. 90.50
Elev.
87.00′
Concrete Dam
Elev. 91.50′
Pit
20′0″ x 60′0″
15′0″ deep
About
Bridge
Surface
Water
Elev. 5.25′
Pump Station
12″ Pipe
Turbine House
Elev. 35.00′

FIG. 16.

along the river bank below and opposite the first basin. Water for it is taken from the first basin.

The water is brought to these basins from the river through 3 030 ft. of tunnels, 460 ft. of galvanized-steel semicircular flumes, about 5 ft. in diameter, 225 ft. of 42-in. pipe, acting as a conduit, and 752 ft. of ditch about 5 ft. wide.

The course of the Eagle River is very crooked, and runs through a canyon in the neighborhood of the plant. At a point up stream, about a mile in a straight line from the plant, the river makes a sharp turn, practically at right angles to its original direction of flow. The bank is of rock, almost perpendicular, and the intake of the settling system is at this point, as shown on Fig. 23. This intake is of concrete, well reinforced, and its top is 3 ft. above high water of usual floods. The position of the grizzly is such that the current during a flood has an opportunity to change its direction of flow completely, and so carry beyond it the greatest part of the driftwood, of which floods bring down very large quantities. The mixtures of concrete used were 1 part cement to 6 parts river gravel; and 1 part cement to 10 to 12 parts river gravel. The gravel found along the river consists of a mixture about in the proportion of 3 parts sand and 5 parts rock, with some silt. The silt is less than 10%, if the location for obtaining the gravel is properly selected. All concrete work in connection with the settling system and around the plant consists of mixtures containing the river gravel just as found, and there have been no failures that can be traced to poor concrete because of the use of this gravel. In all concrete work, large stones have been worked in, to as great an extent as possible. These were first wetted and then dropped into the concrete mixture already in place in the forms, and the mixture was then well worked around them.

At the outlet of Tunnel *A* there is a regulating basin, shown on Fig. 23. The water level in this basin is regulated by boards, *X*, and is maintained at an elevation at which sufficient water is sent through the galvanized-steel flume to the settling basins to supply the pumps, and there is a small quantity in excess. The excess water passes over the boards, *X*, through the wooden flume to the river, carrying with it the largest driftwood which passes through the grizzly. By removing the boards, *X*, the wooden flume serves as a flush-out for Tunnel *A*. Fig. 23 gives the leading dimensions and other information in con-

FIG. 17.—CONCRETE DAM ACROSS GULCH, FORMING FIRST SETTLING BASIN.

FIG. 18.—INTAKE OF SETTLING SYSTEM.

FIG. 19.—AUXILIARY SETTLING BASIN.

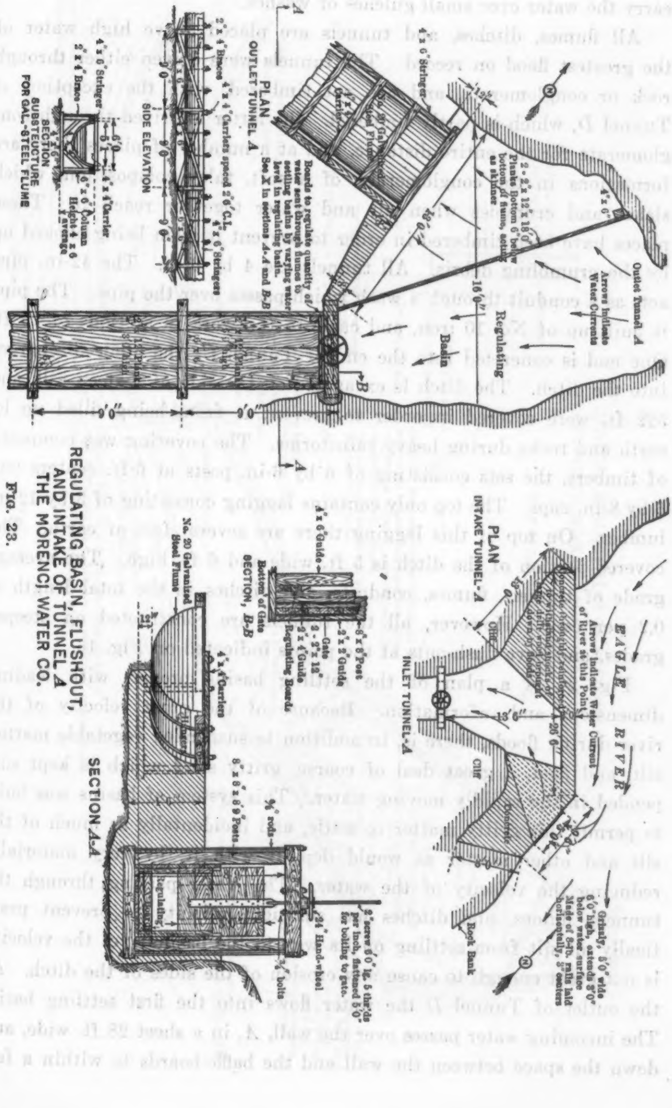
FIG. 20.—SECOND SETTLING BASIN, AND WALL BETWEEN FIRST AND SECOND BASINS.

FIG. 21.—PACKING MATERIAL FOR WORK ON SETTLING BASINS.

FIG. 22.—PACKING MATERIAL FOR WORK ON SETTLING BASINS.

Fig. 21.—Packing Material for Work on Settling Basins.

Fig. 22.—Packing Material for Work on Settling Basins.



REGULATING BASIN, FLUSHOUT AND INTAKE OF TUNNEL A THE MORENCI WATER CO.

FIG. 23.

nection with the design and construction of the steel flumes which carry the water over small gulches or washes.

All flumes, ditches, and tunnels are placed above high water of the greatest flood on record. The tunnels were driven either through rock or conglomerate, and are not timbered, with the exception of Tunnel *D,* which is partly timbered. The latter is driven through conglomerate for the entire distance, and at a number of places there are formations in the conglomerate, of a soft, talcy composition, which slakes and crumbles when air and water together reach it. These places have been timbered in order to prevent it from being choked up by the crumbling débris. All tunnels are 4 by 6 ft. The 42-in. pipe acts as a conduit through a wash which passes over the pipe. The pipe is built up of No. 10 iron, and came to the work in sections of 25 ft. One end is concreted into the end of Tunnel *C*; the other discharges into the ditch. The ditch is excavated on the side of a steep hill, and 522 ft. were covered in order to prevent it from being filled up by earth and rocks during heavy rainstorms. The covering was composed of timbers, the sets consisting of 6 by 6-in. posts at 6-ft. centers and 6 by 8-in. caps. The top only contains lagging consisting of 3 by 12-in. lumber. On top of this lagging there are several feet of earth. The covered portion of the ditch is 5 ft. wide and 6 ft. high. The average grade of tunnels, flumes, conduits, and ditches in the total length is 0.2 per cent. However, all the tunnels are constructed on steeper grades, and have flush-outs at the points indicated on Fig. 16.

Fig. 24 is a plan of the settling basins proper, with leading dimensions and information. Because of the great velocity of the river during floods, there is, in addition to suspended vegetable matter, silt, and float, a great deal of coarse, gritty sand which is kept suspended in the rapidly moving water. This system of basins was built to permit this gritty matter to settle, and incidentally as much of the silt and other matter as would deposit with it, by very materially reducing the velocity of the water. The water passing through the tunnels, flumes, and ditches has sufficient velocity to prevent practically all silt from settling on its way to the basins; yet the velocity is not great enough to cause any erosion of the sides of the ditch. At the outlet of Tunnel *D* the water flows into the first settling basin. The incoming water passes over the wall, *A,* in a sheet 28 ft. wide, and down the space between the wall and the baffle-boards to within a few

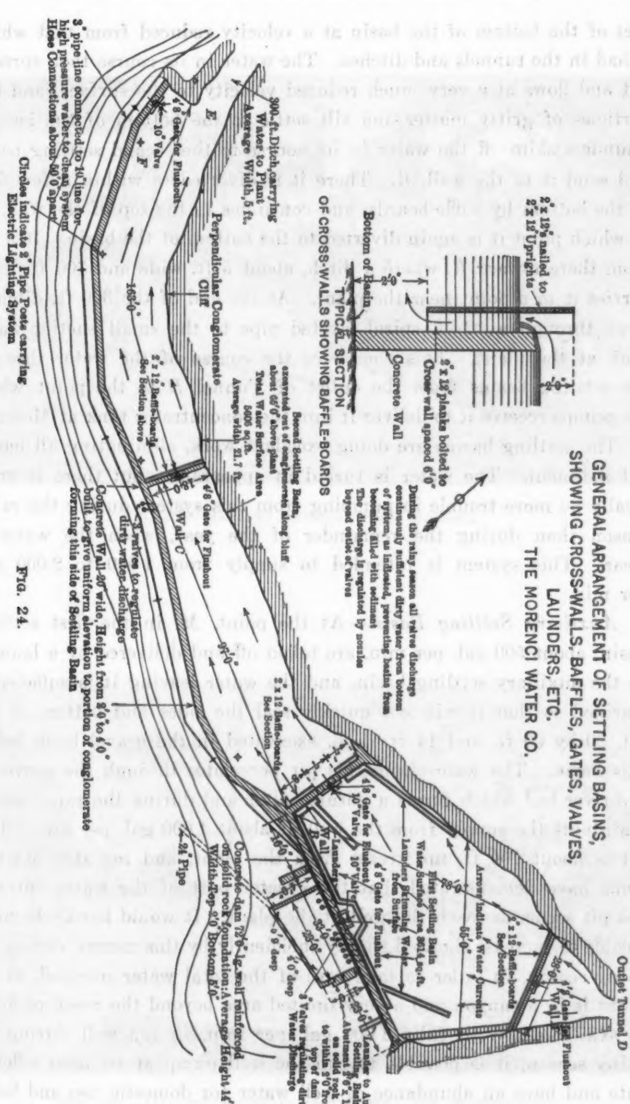

GENERAL ARRANGEMENT OF SETTLING BASINS, SHOWING CROSS-WALLS, BAFFLES, GATES, VALVES, LAUNDERS, ETC.
THE MORENCI WATER CO.
TYPICAL SECTION OF CROSS-WALLS SHOWING BAFFLE-BOARDS
2 x 12's nailed to 2 x 12 Uprights
Bottom of Basin
Concrete Wall
2 x 12 planks bolted to Cross-wall spaced 6'0"
Perpendicular Conglomerate Cliff
300 ft. Ditch carrying Water to Plant Average Width, 5 ft.
4'8" Gate for Flushout
10" Valve
3" pipe line connected to 10" line for high pressure water to clean system Hose Connections about 50' apart
Circles indicate 2" Pipe Posts carrying Electric Lighting System
Second Settling Basin excavated out of conglomerate along the bluff about 60'0" above plant Total Water Surface Area 5605 sq. ft. Average width 21.7 ft.
2 x 12 Baffle-board See Section above
4'8" Gate for Flushout
Wall C
Valves to regulate dirty water discharge
Concrete Wall 20" wide Height from 3'0" to 6'0" built to give uniform elevation to portion of conglomerate forming this side of Settling Basin
24" Pipe
Concrete Dam 7'6" high well reinforced on solid rock foundation; Average Height 14'0" Width at top 2'0" Bottom 5'0"
First Settling Basin Water Surface Area 3014 sq. ft.
Launders skimming Water from Surface
4'8" Gate for Flushout
Arrows indicate Water Currents
2 x 12 Baffle-board See Section
Outlet Tunnel D
Launder to Aux. Settling Basin
Valves regulating dirty Water Discharge
During the rainy season all valves discharge continuously sufficient dirty water from bottom of system, as indicated, preventing basins from becoming filled with sediment. The discharge regulated by nozzles beyond outlet of valves

FIG. 24.

feet of the bottom of the basin at a velocity reduced from that which it had in the tunnels and ditches. The water in its course then spreads out and flows at a very much reduced velocity to the surface, and the particles of gritty matter and silt settle to the bottom of the basins. Launders skim off the water in its course to the second settling basin and send it to the wall, *B*. There it is diverted to within a few feet of the bottom by baffle-boards, and continues to the top of the wall, *C*, at which point it is again diverted to the bottom of the basin. It flows from there toward *E*, where a ditch, about 5 ft. wide and 300 ft. long, carries it to a point near the plant. At the end of the 300-ft. ditch it flows through a 12-in. spiral riveted pipe to the small suction-water tank at the plant. This completes the course of the water through the settling basins from the outlet of Tunnel *D* to the point where the pumps receive it to deliver it into the concentrator tank at Morenci.

The settling basins are doing excellent work, eliminating all harmful sediment. The water is turbid in appearance, but there is practically no more trouble in pumping from this system during the rainy season than during the remainder of the year, when the water is clear. This system is intended to supply from 1 000 to 2 000 gal. per min.

Auxiliary Settling Basin.—At the point, *M*, in the first settling basin, about 500 gal. per min. are taken off and delivered by a launder to the auxiliary settling basin, and the water leaving it is sufficiently clarified so that it will not quickly seal the sides and bottom of the pit, 20 by 60 ft. and 14 ft. deep, excavated in the gravel bank beside this basin. The water from this pit percolates through the gravel of the river bed which forms a natural filter and during the rainy season maintains the supply from the well at about 1 000 gal. per min. This pit is about 700 ft. up stream from the plant, and repeated observations have demonstrated that the greater part of the water entering the pit seems to reach the well at the plant. It would hardly be practicable to increase the well supply sufficiently by this means, during the rainy season, in order to take care of the total water pumped, as the plant is in a canyon and a very limited area beyond the reach of floods is available. With fully 1 000 gal. per min. in the well during the rainy season, it is possible to run the well pump at its most efficient rate and have an abundance of well water for domestic use and boiler feed purposes.

FIG. 25.—HAULING MATERIAL FROM MORENCI.

FIG. 26.—HAULING MATERIAL ON THE HILL OUT OF MORENCI.

FIG. 27.—HAULING MATERIAL ON THE HILL OUT OF MORENCI.

FIG. 28.—SNUBBING A LOAD DOWN A HILL, NEAR THE PUMPING STATION.

FIG. 27.—HAULING MATERIAL UP THE HILL OUT OF MORRIS

FIG. 28.—SKIDDING A LOAD DOWN A HILL, NEAR THE PUMPING STATION.

ELECTRIC LIGHTING SYSTEM AT THE PLANT.

The water discharging from the settling system on its way to the small suction water tank, runs a Leffel water-wheel, to which is belted a General Electric, 14-kw., 125-volt, direct-current generator, at 1425 rev. per min. Sufficient electric current is generated for lighting purposes at practically no cost. A small switch-board, with the necessary instruments, wiring, and lights, completes the equipment for furnishing light for the settling system, plant, bridge, employees' cottages, and buildings. The water-wheel and generator are protected by a corrugated-steel structure with a wood frame, well supported on a concrete foundation.

CONSTRUCTION.

A rough mountain wagon road, more than 6 miles long, constructed by the water company, extends from Morenci to the pumping plant on the Eagle River. A great part of the road is blasted along the rocky hillsides. Out of Morenci, the road runs up a hill requiring a climb 3 400 ft. in length, with grades varying from 17 to more than 36 per cent. Near the plant, there is a continual down grade along very rocky hillsides, with grades of 25 to 40 per cent. On steep grades, snubbing posts, were placed at regular intervals, during the construction period, for the purpose of letting the heavy loads down hill gradually. All material is hauled from Morenci to the plant on wagons. The average loads are about 3 000 lb. A load of this size requires ten horses to make the hill out of Morenci. During the construction period, the heavy pieces of machinery were taken out on a wagon built for heavy hauling, with 4-in. iron axles. Extra rear and front wheels were carried along to replace immediately any that broke down. Some of the pieces of machinery weighed from 15 000 to more than 24 000 lb. each. Twenty horses could make the hill out of camp with a load of 4 tons, and twenty-four horses could manage a load of 5 or 6 tons. It was necessary to use a number of triple blocks, with from six to eight horses pulling down hill, in order to work slowly a load heavier than 6 tons up the hill. At the plant, it was necessary to pack material and supplies around by Mexicans and burros, as in many cases the work was prosecuted at points inaccessible by any other method of transferring material. The cost of transferring machinery from the flat cars at Morenci to the plant, placed ready for the erector, averaged about $52.45 per ton, which is about twice

what the freight per ton amounted to from Milwaukee to Morenci. The 40-lb. and the 54.74-lb., 10-in. pipes were transferred through a tunnel and distributed by wagon to stock piles. In this way, the haul over the steep hill out of town was avoided. From the stock piles, the pipe was "snaked" by horses to the place where it was being laid. The tunnel was too small to permit passing heavy material through it. The cost of delivering the pipe from the cars to the location of the proposed line was $6.80 per ton. The average cost for each 10-in. pipe line laid complete, was $2.20 per ft. The average labor charge, for laying the lines by contract, was 28 cents per ft. for each 10-in. pipe line.

The cost of pumping engines complete, with foundations, auxiliaries, condenser, piping, etc., per 1 000 000 gal. capacity per 24 hours was:

Triple-expansion pumping engines............	$38 500
Cross-compound " "	29 400
Average of all " "	35 500

The cost of the boiler plant, including piping, foundations, etc., was:

Without economizers—per rated boiler horse-power..	$30.50
With economizers " " " " ..	42.50

The total rated capacity of the boiler plant is 640 boiler h.p.

The cost of the pumping plant, including engines, boilers, economizers, piping, etc., per 1 000 000 gal. capacity per 24 hours was $41 500. These figures do not include anything for land, buildings, chimneys, wells, settling system, 10-in. pipe lines, etc., but practically only the items mentioned.

The capacity per pump per 24 hours is 1 500 000 gal., making the total 4 500 000 gal. Investigation will show that, considering the isolated location, the rough country, and the consequent difficulties of construction encountered, together with the high head to be pumped against, the total cost of the plant is reasonable.

DISCUSSION

ALEXANDER POTTER,* ASSOC. M. AM. SOC. C. E. (by letter).—Although dealing with a unique condition, or one which is rarely duplicated, this paper presents a number of phases of general interest to the Engineering Profession. Mr. Potter.

In 1907-08, the writer was called on to design a somewhat similar plant in the Ely copper country, where the pumps were to operate against a static head of 1 020 ft., or a total head, including pipe friction, of 1 400 ft. Precedent with similar high-lift pumping machinery was very limited at that time, and there was considerable doubt as to the practicability of the enterprise.

The author is to be complimented for setting forth in such detail the difficulties experienced in operating the pumps under such unusual conditions of service. The performance and efficiency of the pumping units, as proved by test, seem to be highly satisfactory. However, it is not clear from the paper whether, during the test, the engines were properly charged with the steam used by the auxiliaries, such as the condensing apparatus, air pump, etc. Furthermore, it appears that during the test the engine was not charged with the additional fuel which must have been used in superheating the steam to 142° Fahr. instead of the 100° Fahr. called for under the guaranty.

The practice is sometimes resorted to, in order to obtain the highest duty, of making the trial duty test under conditions more favorable for a low steam consumption than can be followed in practice, such as maintaining high vacua and high degrees of superheat, and of not charging the engine during the test with the steam consumed by the auxiliaries, which are evidently run at their maximum capacity in order that the steam consumption of the engine proper shall be a minimum. Furthermore, in a recent test on a large installation, made under the direction of a well-known testing laboratory, not only was this done, but no account was taken of the additional coal consumed in forcing the boilers to obtain the higher degree of superheat. In this manner, the result obtained was much in excess of the actual duty of the pumping unit.

In the case referred to by the author, the specifications call for a guaranty of 1 000 lb. of dry steam supplied to the engine under definite conditions; yet, during the test, the steam was furnished with 42° Fahr. more superheat, and it is not apparent that any deduction was made therefor. Neither is it clear that the engine was charged with the steam used by the other auxiliaries.

Recently, the writer was requested to report on a test which had been conducted on engines, the performance of which was guaranteed at 165 lb. steam pressure and 125° superheat. During the test period,

* New York City.

Mr. Potter. a superheat of from 240° to 250° Fahr. was maintained, as well as a much higher vacuum than it is possible to maintain under the most favorable operating conditions. Furthermore, in order to obtain the highest mechanical efficiency of the pump, which is of the centrifugal type, it was operated during the test under 4.5 ft. less lift than it would have to be operated against in practice. This was accomplished by admitting water to the suction well, thereby keeping it artificially at a higher level. The duty was based on the steam actually delivered to the engine. The records show that, during the test, the steam for the auxiliaries, amounting to 13.6% of that consumed by the engine proper, was ignored in computing the duty. The percentage of 13.6 includes a small quantity of steam leakage, etc., not chargeable against the engine. Furthermore, in computing the duty, no account was taken of the additional coal consumed in forcing the boilers to obtain the high degree of superheat.

Results obtained in this manner are misleading, to say the least. The actual operating duty at this particular plant for the last year averaged less than 50% of that alleged to have been obtained at the test.

The author's statement, that the mechanical efficiency of the triple-expansion pumping engine, as calculated by dividing the actual duty by that obtained from the indicated horse-power, was 93%, would seem to be such a remarkably good showing, under the conditions existing at this plant, that the writer would like to know whether pump slippage was taken into account.

The statement, that the average quantity of water pumped, during 1913 and 1914, was from 95.02 to 95.3% of the plunger displacement, appears to be very remarkable, under the conditions obtaining at the plant.

The statement, that it was not possible to retain sufficient air in the air chambers to be of benefit is very interesting. The writer does not agree with the author's conclusion, that the absence of difficulties in running without air chambers was due to the formation of small air pockets at the summits, caused by the irregularities in the 10-in. pipe line conveying the water to the reservoir. It is his opinion that if the water would absorb the air from the air chambers of the pumps, the accumulation of air along the summits of the line would also be impossible, except at those points near the upper end of the force main where the pressure is very small. In the writer's opinion, one of the reasons that difficulties were not experienced on the pipe line was the fact that, to a very great extent, the elasticity of the wrought-steel pipe absorbed the effects of water-hammer.

Mr. Duncan. LINDSAY DUNCAN,* M. AM. SOC. C. E. (by letter).—Although the milling of low-grade ore requires from 5 to 10 tons of water per ton

† McGill, Nev.

of ore, the water supply is seldom given sufficient consideration in the location of ore mills. Mr. Duncan.

One would infer that at Morenci the mills and town struggled along for some years with an inadequate supply of very poor water, until they finally combined their resources and built the very complete plant which Mr. Du Moulin has described.

Off-hand, it would seem to the writer that a supply of clear water for the pumps could have been obtained readily by driving a filtration gallery under the river bed. This gallery could have started from the shaft, which would have been available as a suction sump. The writer supplemented the water supply of the Nevada Consolidated Copper Company, at McGill, Nev., in this manner, by driving a tunnel 1 150 ft. across the valley of Duck Creek, and obtained a flow of 1 500 gal. per min. from a depth of 50 ft. The overburden was composed of alternate layers of clay and gravel, and these were broken up by removing the lagging from the roof of the tunnel at 50-ft. intervals and starting "runs" which caved the ground up to the surface.

The low vacuum reported seems to the writer to be due to air leakage into the condenser. Minute openings in the cast-iron condenser shell, or imperfect joints in the exhaust piping, will affect very seriously the vacuum of a condensing engine.

The writer also believes it to be bad practice to introduce an oil separator between the exhaust nozzle and the condenser, as it is bound to reduce the vacuum available at the engine. It would be better to filter the condensate and put in a surface blow-off, or skimmers, in the boilers.

In the absence of any statement, the writer assumes that the vacuum was measured at the exhaust nozzle, where the deleterious effect of the oil separator would be felt.

Considering the low vacuum, the duty obtained by these engines is truly remarkable, and the writer would be interested in additional details of the tests. A statement of the number of gallons of water pumped per pound of oil burned, together with the characteristics of the oil and the method of measuring the water pumped, would round out the paper, particularly if the author were able to give the duty per pound of oil of the new plant under test conditions and also after it had been run for several years. The writer's experience with high-grade Corliss engines leads him to believe that a 30% increase in steam consumption is to be looked for in a plant 10 years old.

H. Hawgood,* M. Am. Soc. C. E. (by letter).—The Morenci pumping plant is an instance of special construction to meet special conditions in an extremely rough and precipitous country. The author and his associates are to be congratulated on their successful solution of the inherent difficulties of the situation. He would add to the Mr. Hawgood.

* Los Angeles, Cal.

Mr. Hawgood. completeness of his valuable paper by giving the ordinary revolutions per minute of the pumping engines, and also the quantity of water pumped through the 10-in. line with a friction head of 175 ft., as stated.

The writer has visited the plant on two or three occasions. The pumps run smoothly, without jar, notwithstanding the absence of air chambers. When the difficulties of charging and re-charging air chambers with air at about 800 lb. per sq. in. are considered, their omission is sound. It is probable, however, that their introduction at points of moderate pressure on the pipe line would mitigate the water-hammer caused by cyclic variation in rate of flow, due to uncushioned contact with crank-driven plungers.

The water-hammer in the last 5 000 ft. of the line was rhythmic and heavy. The time interval between blows, however, did not appear to be uniform or entirely in unison with the pump speed. This may have been due to the numerous intervening summits on the pipe lines affording opportunity for air pockets.

The relief given by the inward-opening check-valves, described by Mr. Du Moulin, proves the correctness of his diagnosis as to the cause of the local water-hammer.

It is suggested that free discharge into small open tanks at suitable elevation at the highest point on the line, and steady gravity flow from there to the storage tanks would eliminate the hammer troubles and obviate the flow disturbance which must be created by the introduction of air. Proper balance between the outflowing gravity water and the inflowing pump water could be maintained by one of the several simple devices for that purpose. Air chambers at the critical high points in the 5 000-ft. stretch might accomplish the same results.

The settling system is particularly interesting. The problem of creating along the sides of a box canyon a system sufficient to clarify the storm-waters has been worked out in a very practical manner. The material increase made in the available supply, and the increase in efficiency and life of the pumps obtained by the introduction of the settling plant, certainly warranted its cost.

The filtration through the river gravels of the supply destined for domestic use has its counterpart in the growing practice in Southern California of spreading the excess winter stream flows over the gravel débris cones at the canyon mouths. The underground storage thus accomplished is available later for summer pumping. The average infiltration rate is about 2 sec-ft. per acre. Gravelly lands covered with brush are generally more absorptive than the bare gravel and boulder beds of the streams themselves. This phenomenon is due to sealing by silt. Violent floods tear up the beds and renovate the absorptive capacity. It would be interesting to hear from Mr. Du Moulin as to the methods he has adopted to obtain new infiltration surfaces.

W. L. Du Moulin,* Assoc. M. Am. Soc. C. E. (by letter).—The duty test of the cross-compound pumping engine was made within 4 months after that engine was started, and was in the nature of an acceptance test, in which the conditions were as similar as practicable to those called for by the specifications. Mr. Du Moulin.

The duty test of the triple-expansion pumping engine, however, was made after it had been run several years, and was more for the purpose of determining the duty that it might develop under favorable test conditions. For this the Water Company employed a responsible testing engineer. Everything that was essential was done in order to gain accurate results, although possibly some slight refinements were disregarded, as it was not an acceptance test; but, neglecting to make such refinements did not, in the writer's opinion, affect the results essentially. The data show what was accomplished under the conditions indicated. As the auxiliaries are of the attached type, the steam required by them during the test was naturally included in that charged to the engine. No correction was made for the 42° Fahr. higher superheat obtained than was called for under the guaranty. With this correction, the duty would have been in the neighborhood of 177 808 000 ft-lb. No allowance was made for pump slippage. The duty was based on plunger displacement. Tests on these pumps have shown the slippage to be between 1½ and 2 per cent. Naturally, the pump was carefully examined before the test, and all packing and valves were put in first-class condition, with not the least suspicion of a leaky valve in the pump. Under such conditions, it is generally customary not to take the pump slippage into account. Moreover, the quantity of water leaking past the plungers was so small that it was of no practical consequence. This small quantity goes out under the working pressure, and represents work just as though it escaped through the discharge pipes. The plungers, moving against water pressure, do work for the full length of their stroke. With a reciprocating pump in good condition, as was the case in each instance during the Morenci tests, the duty, based on plunger displacement, is as accurate and satisfactory for all practical purposes as though slippage had been taken into account, especially when weir measurements and the Venturi meter usually do not check each other within 2 per cent. In the writer's opinion, duty based on plunger displacement is really the only reliable means of comparing the performance of large reciprocating pumping engines built by different manufacturers. It reduces the matter of personal equation very nearly to the minimum. Of course, in comparing the duty of a reciprocating with a centrifugal pumping engine, the slippage of the former should be ascertained and an allowance made, as the duty of the centrifugal pump is usually measured in foot-pounds in the water. However, in

* Morenci, Ariz.

Mr. Du Moulin. such case, the error would not be great enough to affect the result materially if slippage were not taken into account.

In conducting a duty test, the object is either to determine whether a pumping engine comes up to its guaranty, or to find out what duty it is capable of developing under favorable conditions. This latter information is particularly of interest from an engineering standpoint. In the tests of the Morenci engines, the aim was to obtain fair results; and, as stated, no attempt at "boosting" conditions, and consequently results, was made.

A purchaser generally knows what his operating conditions will be, and is able, therefore, to give the necessary information, showing under what conditions the pumping engine is to work, with a fair degree of accuracy. Sometimes a bad "guess" is made, however, or again it frequently happens that unforeseen circumstances compel changes after a pumping engine has been ordered, so that the actual operating conditions vary materially from those for which it was designed. As a consequence, the average duty developed by the engine under operating conditions will fall far below that obtained under test conditions. The writer is of the opinion that this fact should not count against the engine or its designer; for, if the average conditions were more favorable, a better everyday duty could be undoubtedly maintained. A bad "guess" or altered or unfavorable operating conditions may account to some extent for the very large discrepancy in the case cited by Mr. Potter, between the everyday performance and the duty test, as well as the fact that the result of the test was "boosted" by methods which were far from equitable. In the Morenci case, the actual operating conditions varied from those anticipated when the triples were ordered. The cross-compound was put in later, and the builders were able to design this engine to conform more closely to the everyday conditions. This, together with the better vacuum that this engine carries, accounts perhaps for the fact that its everyday performance is not only equal to, but slightly better than, that of the triples. There is no doubt that, with a better vacuum and more favorable operating conditions, the performance of the triples would be the better.

For 1912, the actual water pumped was 94.5% of the plunger displacement. The improvement in the efficiency of the pump end has been gradual, and has been the result of changes mentioned in the paper, and of detailed attention given this matter during a period of some 5 years. The report on the property of The Morenci Water Company, made by Mr. C. E. Sloan, Engineer for the Arizona Corporation Commission, in 1913, contains the records to that date. Mr. Sloan and his assistants made a careful examination of the property and the records, and his report substantiates statements made by the writer in regard to pump end performances. The improvement in the

performance of the pump end of the Morenci pumps has been about 18 to 20% since they were started. The maintaining of this improved performance is due principally to the detailed attention given to plunger packing and valves. It is the aim to maintain them continually in first-class condition. The pressure is very great, and leaky valves soon do much damage. As soon as a leaky valve is noticed, it is immediately replaced by a good one. This is not the general practice in pumping plants. In fact, the care of valves is a feature that is neglected. In most plants, the practice seems to be of an arbitrary nature, the valves being overhauled once every year or two. The writer has visited a great many pumping plants of all sizes, and it is his opinion that in most of them a very large saving can be effected by giving more detailed attention to the valves, thereby reducing excessive slippage and leakage, and the loss represented thereby. In one instance, the slippage was more than 30 per cent. Overhauling valves more frequently will fully pay for itself in the improved performance of the plant. Mr. Du Moulin.

Mr. Duncan's description of the filtration gallery that he constructed to obtain additional clear water is very interesting, and, under the right conditions, is a very feasible method, but it would not be practicable at Morenci. The formation of the bed of the Eagle River though similar, differs from that of Duck Creek at McGill, Nev. It consists of a river gravel but a great part of it contains more silt and there are also layers of silt running through the bed. These layers form quicksand when saturated with water, and are practically impervious when relatively dry. A hole was sunk to a depth of 21 ft. in the river gravel on the bank of the stream less than 8 ft. from the running water on the surface. The gravel was fairly moist at that depth, but no water in quantity collected in the bottom, thus plainly demonstrating to what extent the layers of silt seem to retard the percolation of the surface water. A tunnel was started across the river bed at a depth of about 40 ft. After proceeding some distance, it caved in, breaking up the ground to the surface and causing quite a depression. However, the silt sealed up the broken ground so completely that the water from the surface was prevented from filtering through in quantity. As the silt seals up the river bottom and sump hole sides and bottom so quickly during the rainy seasons, the method described by Mr. Duncan would be impracticable at Morenci. During the rainy season, the river is generally almost continuously muddy, and brings down the exceptionally large quantities of heavy, thick silt which settle quickly and do the damage. The writer's experience has demonstrated that, in general, the conditions at Morenci during the rainy season are not favorable for any kind of filtration scheme, unless the silt is first allowed to settle out of the water to a great extent by passing it through settling basins. As the settling system

Mr. Du Moulin. water is suitable for mill purposes, and there is sufficient well water for all other purposes, it was not necessary to incur the additional expense of a filtering system. Sufficient work was done toward the development of additional well water to demonstrate that it was practicable. The cost, however, would be very great, because of the isolated location and the expense of getting the necessary supplies to the plant. The settling system was the most practical and the least costly way of obtaining a satisfactory water. It had the additional advantage of eliminating the cost of lifting water out of a deep well or sump, as the settling system water flows by gravity to the pumps. Also, its cost of maintenance is very low, in fact inconsequential.

Mr. Hawgood's description of the method of obtaining underground storage in Southern California is very interesting. The observations of the writer bear out Mr. Hawgood's remarks on the effect of violent floods in increasing the absorptive capacity of the river bed. There have been several such floods on the Eagle River during the past 6 years, caused by rains and melting snows in the mountains, with the result, in each instance, that the yield of the well was materially increased during the succeeding months, and until the next rainy season. As soon as that season arrived, the small floods and muddy water sealed up the river bottom, with a consequent decrease in the well supply, and, in order to maintain the well at a certain capacity, infiltration surfaces had to be used. For reasons given in the paper, it was impracticable to use this method during the rainy season to obtain a sufficiently large quantity of clear water for all needs. Before the construction of the settling system, the infiltration surfaces were limited to a series of shallow pits that were cleaned very frequently with pick and shovel. This was tedious and rather expensive. Since constructing the settling system, the area of infiltration surface is limited to the single large pit mentioned in connection with the auxiliary settling basin. Because of the better water, this surface need only be renewed once or, at the most, twice a year. This is done by washing the silt down the sides of the pit with high-pressure water, which is available, and pumping out the thick muddy water from the bottom with several bilge pumps.

Careful inspection of all exhaust piping and exhaust connections has been made frequently, and no air leaks have been found of sufficient consequence to account for the low vacuum. All engines are equipped with recording vacuum gauges, and any variation in the vacuum is investigated. Since the paper was written, a jacket of galvanized iron has been placed around the condenser shell, with a space of 1½ in. between the latter and the jacket. Through this jacket, around the outside of the condenser, water is circulated, the space being kept full of circulating water. This arrangement caused an improvement of 3 in. in the vacuum, showing that the area of effective

cooling surface is insufficient. The fault lies principally in the design of the condenser. The vacuum was measured at a point between the oil separator and the condenser inlet, close to the latter, a mercury column being used. All boilers are equipped with surface blow-offs. Mr. Du Moulin.

The writer notes with interest Mr. Duncan's remarks in regard to the fact that his experience indicates the expectation of 30% increase

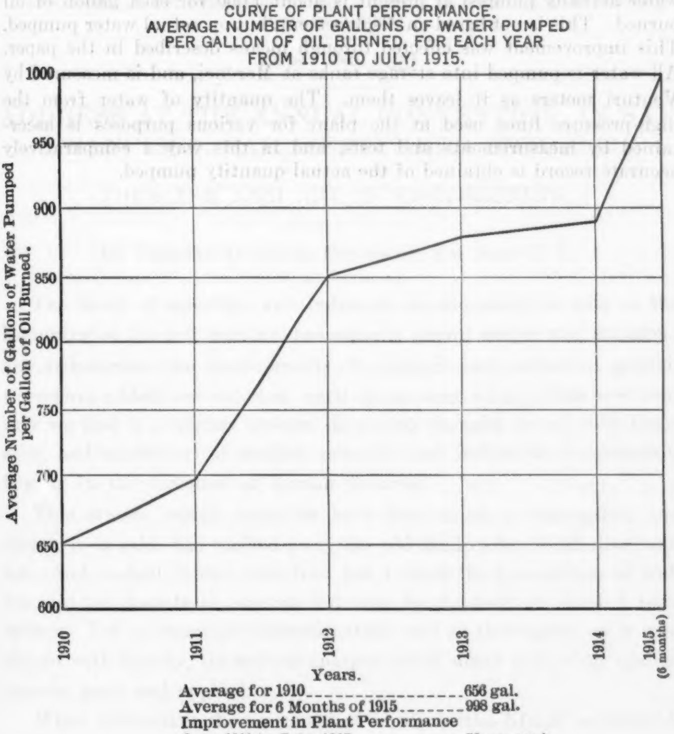


FIG. 29.

in steam consumption in a plant of high-grade Corliss engines 10 years old. In a power plant with high-speed machinery, the writer believes that this might be the result; but low-speed pumping machinery, with the uniform load and service resulting from pumping into a reservoir, should not show with age much variation from its original steam economy, if properly maintained. One of the Morenci

Mr. Du Moulin. triple-expansion pumping engines has been in service almost continually for 6 years; and its performance to-day is, if anything, better than it was originally.

The curve and data on Fig. 29 give the history of the plant performance during the past 5 years. The improvement from the end of 1910 to July, 1915, has been 52%, and the average number of gallons of water actually pumped at present is about 1 000 for each gallon of oil burned. This is not based on displacement, but on actual water pumped. This improvement was effected through means described in the paper. All water is pumped into storage tanks at Morenci, and is measured by Venturi meters as it leaves them. The quantity of water from the high-pressure lines used at the plant for various purposes is ascertained by measurements and tests, and in this way a comparatively accurate record is obtained of the actual quantity pumped.

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TRANSACTIONS

Paper No. 1351

ADDRESS AT THE ANNUAL CONVENTION, IN SAN FRANCISCO, CAL., SEPTEMBER 16TH, 1915

IDEALISM AND ART IN ENGINEERING

By Charles D. Marx, President, Am. Soc. C. E.

The brook of scientific and industrial development, so tiny at the beginning of the last century, has steadily grown deeper and stronger; new tributaries—the developments of scientific and industrial specialties—have added new volumes, until at the beginning of this new century we find it a mighty stream, deep with thought, broad with liberality, and symbolical of modern scientific and industrial development, flowing in the direction of human progress.

This stream, which scientists have done much to strengthen and direct, it is said, has washed away the old landmarks of idealism and art. Sad, indeed, if this were true, but I think the foundations of both are laid too deep to be scoured out even by the flood of which I have spoken. Let us examine dispassionately, and as thoroughly as is consistent with brevity, the serious charges which many still bring against science, pure and applied.

What determines the existence of ideals in the life of an individual—in that of a nation? The man of the highest ideals is not a man of words, but a man of action. He is not content, as Dean A. W. Smith of Sibley College said in his beautiful tribute to Ex-President A. D. White, to be a mere seer of visions.

"Some men who see rare visions rest content
To see them and to let them fade away;
Not so with him; to him the vision meant
The call to toil to make the vision stay."

Numberless are the instances in the past as well as in the present where poets have sung and clergymen preached self-denial, self-sacrifice, humility and service, the essentials of an ideal life as typified in Christ. Numberless too, I am sorry to say, are the instances in which the avowal of such ideals has not carried in its wake a living up to them. The possession of ideals, or rather the being possessed by them, must show itself in a man's life, and is in a large measure independent of his occupation. I am fully aware that in making this statement I am running counter to Mr. Ruskin, for he inveighs strongly against what he calls the "thought-killing work of the masses of our laboring classes", and for a good deal of this work the Profession of Engineering is responsible. Though not blind to the fact that ten hours a day of incessant toil in digging a trench or running a machine will not develop a man as we should like to see him developed, I still claim that if in such a case the ideals in a man's breast are killed, it is owing to the amount of work and not to its character. Physical exhaustion has become so great, that mental exertion is out of the question, and man is brought to the level of the brute. But even under these most unfavorable conditions we have numberless instances of men and women leading ideal lives—heroes and heroines living and dying unsung. Now and then a brief notice, such as I cut out of one of our large dailies, passes under your eye:

"Napoleon de Montague, a miner, was killed yesterday in Lance colliery in Plymouth, while endeavoring to save his fellow-workmen from a terrible death. He had fired a shot and ran behind a pillar, when the shot exploded, and the flash set fire to some gas near the roof of the chamber communicating with the main gangway. Realizing that the fire might spread in a moment through the whole mine, de Montague tore off his coat and smothered the flame. Just as the fire was extinguished the roof of the chamber, loosened by the blast, fell upon and killed him. His act probably saved many lives."

You give this notice but a passing glance. It lacks the glamor of romanticism. How much nobler, how much more ideal it would seem to our romantic friend, if it were the deed of valor of a knight errant who fell defending some imaginary slight cast upon some imaginary honor. This, however, is only a case of self-sacrifice in the utilitarian occupation of coal mining, illy suited for poetic garb and hardly fit for polite society. How can idealism and utilitarianism mate? We

all travel unhesitatingly on land and on water. We confide our bodies, which we generally deem of more value than our souls, to car or boat. Have you ever fully realized, in reading accounts of the comparatively few accidents that do occur, that most of them are accompanied by some such statement as: "the engineer reversed his lever, stuck to his post, and was found dead under his engine", or "the pilot stood at the helm until the burning boat was beached"? These cases are so numerous that it seems a platitude to speak of them. Acts of heroism as great as any that were done in the past, as noble as any, for they are performed, not in the much sung vocation of "killing" our neighbor, but in that "despised" one of "saving" him. In a recent number of *The Forum,* H. M. Chittenden, Brigadier General, U. S. Army (Retired) and an honored member of society, has an article on "Peace and Heroism" the reading of which I recommend most heartily. It is a splendid tribute to the heroes of civil life. I really should quote the article as a whole, did space permit, but I must confine myself to a brief extract:

"But while war, in the very nature of things, abounds in opportunities for valorous exploits, and its every deed is written large on the page of history, the humbler and quieter sphere of private life affords even more and keener opportunities for the display of true heroism. The physician or nurse, who voluntarily goes into a plague-stricken district, the miner who braves the fire-damp to rescue his imprisoned fellows, the crew who stand at their posts while their vessel is sinking, the fireman who scales a tottering wall to save a human life, the patrolman who enters a den of desperadoes at imminent personal risk—whoever in the pursuance of duty, no matter how humble, subordinates his personal safety to that of duty—is as much entitled to the commendation of heroism as a soldier who does his duty in war can possibly be. These opportunities for heroic deeds are everywhere with us and always will be. They may lack the glamor of war and go unblazoned to the world, but the very humbleness of their status enhances, if anything, their heroic quality."

And as Dr. Jordan has well put it, in speaking of the work of a member of this Society, who has had charge of the relief of six millions of Belgians against "democratic famine working day and night", "this was a problem of infinite dimensions". But an engineer, "born with a streak of idealism" and with great executive ability, tackled this problem and solved it. Solved it so well that one of his colleagues in the work writes: "It's often been said that our American Commis-

sion is the hope of Belgium. Revise that—it's Hoover, that's the hope of Belgium."

Engineering destructive of idealism! Little study has he made, who advances this claim, of the close association between religion and engineering in the past—between religion and engineering in the present. The monks of the Twelfth Century who toiled patiently placing stone on stone in the bridges which they erected, saw more clearly than do many of the present century the relation between paths of communication and the spread of religion. They saw and felt that the greater the number of timber and stone ties with which they bound adjacent countries together, the greater would become the number of spiritual ties, the more would recede the causes of war, the nearer would approach the day of "on earth peace, good will toward men". And toward this same end the men in the Profession of Engineering have been laboring. It is true that at the present time, these ties, which we have been so long in constructing are being severed, but for the severing of these ties the Profession of Engineering is not responsible.

Now let us turn in detail to various branches of our Profession, and see if the practice of them is likely to be destructive of idealism. The Mosaic code of laws shows that the health of the Jewish population was a matter of such supreme interest that special legislation on that subject seemed warranted. The Greeks and particularly the Romans recognized fully the moral value of personal cleanliness; but it has been left to our country to show clearly and on the basis of scientific study, the interdependence between sanitation and disease, sewerage and crime. Münsterberg has said, "Hygiene can prevent more crime than any law". Speaking before the Massachusetts Conference of Health Officials, Dr. Charles W. Eliot, President Emeritus of Harvard, addressed them in part as follows:

"The progress in knowledge of preventive medicine made during the past fifty years, and in applications of that knowledge in social practice, has been the most cheerful phenomenon in the recent history of civilization. The new applications of physical forces—heat, light and electricity, which mankind has learned to use in its conflict with nature—have proved to be highly beneficent in the field of preventive medicine. Civilized communities have been enabled to make their water supplies, food supplies and drainage systems safe, and to contend with

unexampled success against formidable pestilences, the common communicable diseases, and the bodily ills which attend urban life and the factory system."

In this work of preventive medicine the sanitary engineer has borne his full share. As we read these inspiring words of Dr. Eliot our thoughts turn at once to that monumental work in sanitation carried out on the Panama Canal under the direction of General Gorgas, and with the hearty co-operation of the engineering staff. Under disheartening difficulties, and with sacrifice of personal comfort, yes, at the risk of their lives, the men labored, who made the Isthmus a place fit for the white man to live, and thereby made the construction of the canal, which means so much to mankind, possible. Honor, deserved honor, has come to him who directed this splendid piece of sanitary engineering, but I desire to put on record here the appreciation of this Society for the men who stood by. Is it reasonable to suppose that the engineers who did this work, who made these sacrifices, were not the highest type of men, *i. e.*, practical idealists? I could go on and specify instance on instance in which modern sanitary engineering has achieved results similar to those already brought before you. Most of you have no doubt read Dr. Shaw's extremely interesting papers on municipal reforms in London, Glasgow, Berlin, Paris, Vienna, Naples, and even ancient Rome. That these last named cities should also be attacked by the modern spirit of reform seems to our romantic friends proof conclusive that "ideals" have perished. We are called iconoclast engineers, utilitarians, non-respectors of tradition, antiquity, and picturesqueness. Iconoclasts, yes; we plead guilty to the charge; reformers usually are. Utilitarians; that too is an accusation that we cannot deny; but is an act less noble for being useful? Non-respecters of tradition and antiquity! Again we plead guilty, if tradition stands for error, and antiquity for decay. Destroyers of the picturesque; a general denial cannot be entered against this charge either; and in some cases, I am free to confess, too little justification can be shown for the disregard of the picturesque. This is a point I shall touch upon more fully later; but with special application to the cases cited above and similar ones, what is picturesqueness but the diseased condition of structures and their surroundings? What is this vaunted love of the picturesque in many cases, but a selfish and thoughtless appreciation of surface appearances? Selfish and thoughtless, I say, because the few

are willing to sacrifice the weal and woe, the health and happiness of the many, in order that they may feast their eyes on narrow winding streets, on quaint gables, far-reaching eaves, small, curiously leaded panes of glass. They are taken in by the surface appearance of things. But little heed give they to the squalor and dirt, the misery and sickness existing in these picturesque quarters. If but sunshine stream merrily through the broad, light panes in their houses, if but their sanitary appliances are of the best, what thought give they to the life of those who are huddled into those picturesque quarters? Fondly their thoughts turn back to the heroes of the past, to them their hearts go out. The men of the present become heroes to them only as they too become a part of the past. Let me ask you: Who then is the idealist? The man who, probing into the sore which has so little surface indication, finds its deep-rooted seat, and skilfully uses the knife; or the man who, misled by these same facts, applies a surface dressing and allows the sore to eat into the body? To the thinking man the answer is simple. Perhaps you will grant now that one branch of engineering, at least, namely, sanitary engineering, and idealism, are not only not incompatible, but that they are almost inseparable. And what I have shown somewhat fully for this branch of the Profession, can be shown as well for the many other branches.

Take irrigation engineering, for instance, the possibilities of which are only beginning to be realized in this country. The massive dams now building in the fastnesses of the Sierras, the Rockies, and the other mountain ranges from which spring our rivers, will store safely behind their broad backs the precious water which has long run to waste. Thousands of miles of ditches, of pipes in iron, steel, and wood, will lead this water to the thirsty soil. The wonders which Mother Earth, in gratitude for this quenching of her thirst, accomplished are, to many of you, wonders no longer. Is it likely, I again ask, that the men carrying out these works see in them but the piling of one stone upon another, the digging of so many feet of trench, the laying of so many feet of pipe? Believe me, these black cast- or wrought-iron cylinders stand for more than this to the true engineer. He realizes that with every water or drain pipe well laid he is bringing prosperity and happiness, health and vigor, where before existed poverty and misery, sickness and languor. Perhaps the most wonderful instance on record in modern times of the far-reaching effects of irrigation engineering is

found in Egypt. In an article on the regeneration of Egypt, by the former librarian at Stanford, Mr. Woodruff (now Professor of Law at Cornell), he unhesitatingly and justly, I think, attributes a large share of the credit for this "new birth" to the work of the English engineers. Mr. Woodruff says:

"The history of the English in the administration of Egypt for the past nine years is the record of the return to health, strength and prosperity of a country that has been bled and starved almost beyond resuscitation. And yet there has been little romance in this restoration. It is chiefly a story of common sense, honesty and straightforward hard work."

Mr. Woodruff then states more in detail the works of construction and repair carried out by the English engineers and pays the following tribute to the latter as men of high ideals:

"A word must be said as to the character of these engineers who have been foremost in the redemption of Egypt. They have had to contend with vested abuses on every side, learn the spoken Arabic of the common people and overcome religious prejudices and superstitions which, as Balzac says, are the most indestructible form of human thought. Each of the present irrigation inspectors travels his district again and again, often on foot, suffering much hardship and seeking the shelter of the humblest mud-huts lest by accepting the entertainment of the wealthy proprietors he be suspected by the poorer natives of having been bribed. In former days a poor man was completely at the mercy of his rich neighbor and of the corrupt native inspector, who unless bribed, would not open a sluice at a critical time for the crops. Now the poor no longer have to bribe for water, they have confidence in the English inspectors and have learned that petitions will be listened to and wrongs redressed."

And to this strong tribute Viscount Milner in his book on "England and Egypt", has added even a stronger one in his chapter on "The Struggle for Water". I have never read a more interesting or appreciative description of the work of the engineer. I can but recommend to you the reading of it, and must content myself with but a brief extract from this chapter, and the retelling of a story which shows the appreciation in which our brother engineers are held. First, Viscount Milner says:

"The longer I remained in Egypt, and the more I saw of the country, the more clear it became to me that the work of these men has been the basis of all the material improvement of the past ten years."

Then he continues:

"Only one case in point to conclude with. It is a story which I think I have seen in print before, but it is so remarkable that it will, perhaps, bear repetition. In the bad year 1888, when, as has been stated, the Nile flood was an exceptionally poor one, there was a large area in the province of Girga which was threatened, like many others in Upper Egypt, with a total failure of the inundation. The canal which ordinarily flooded this particular district was running at a level at which the water could not possibly spread over the fields, and many thousands of acres seemed doomed to absolute barrenness. A cry of despair arose from the whole neighborhood—What was to be done? One of the English Inspectors of Irrigation, who happened to be on the spot, promptly determined to throw a temporary dam across the Canal. The idea was a bold one. The time was short. The Canal was large, and, though lower than usual, it was still carrying a great body of water at a considerable velocity. Of course, no preparations had been made for a work the necessity of which had never been contemplated; but the Inspector was not to be daunted by the apparent hopelessness of the undertaking. Labour, at any rate, was forthcoming in any quantity, for the people, who saw starvation staring them in the face, needed no compulsion to join gladly in any enterprize which offered them even the remotest chance of relief. So the Inspector hastily got together the best material within reach. He brought his bed on to the Canal bank, and did not leave the scene of operations, night or day, till the work was finished. And the plan succeeded. To the surprise of all, the dam was, somehow or other, made strong enough to resist the current. The water was raised to the required level, and the land was effectually flooded.

"The joy and the gratitude of the people knew no bounds. It was decided to offer thanksgiving in the Mosque of the chief town of the district, and the event was considered of such general importance that the Minister of Public Works himself made a special point of attending the ceremony. But the enthusiastic population were not content with the presence of the high native dignitary. They insisted that his English subordinate also should be there. They were not willing to give thanks for their deliverance without having amongst them the man who had wrought it. Every one knows how deep a prejudice exists in Mohammedan countries against the presence of a Christian in a mosque. In the great tourist-visited cities of Egypt this feeling is wearing off, but in the country districts it is as strong as ever. In those districts it is an unheard-of thing that a Christian should be present at a religious ceremony—more than unheard-of that he should be present at the instance of the Mohammedan worshippers themselves. But in this case the universal feeling of thankfulness and admiration was too strong for the most deeply-rooted fanaticism. For the first

time, doubtless, in the history of that neighborhood, an Englishman and a Christian was allowed, and even compelled, by the natives, to take part in a solemn function of their usually exclusive and intolerant faith."

But we need not turn to foreign countries to find work of the irrigation engineer worthy of commendation. Our own Reclamation Service has made a record of which we may well feel proud. Here too the men, with a devotion to the cause of mankind, with loyalty and steadfastness, have given a service, the value of which is not yet fully recognized. These are the men of whom Senator Newlands said, in the United States Senate, on February 17th, 1910:

"We have had one of the most capable and honest construction services organized that has ever existed in the history of this country. The Committee of the Senate on Irrigation has been engaged during the past year in visiting these various works, and not a whisper of corruption has reached them. It has been a work conducted with rare integrity and with rare speed. In carrying out the Egyptian irrigation projects, both works and settlers were financed. If our projects are to rival them in success, our Government must adopt that policy. The engineers have done their work well; they are the men of whom Chief Engineer Davis said: 'Their chief tie to the service is not the matter of salary, but interest in their work and loyalty to it and the belief that they are appreciated'."

Again, I ask, is such work destructive of idealism, are such men lacking in ideals?

In railroad engineering, think you that the men who through virgin forests and sandy deserts, through miasmatic swamps and rocky cañon, across rivers and over mountains, carried the steel bands that now tie mankind so closely together—think you that these men were engaged in an occupation likely to kill their ideals? When the final balance is struck, I warrant that the debit will not be on the side of this grand army of peace of the present, as compared with the armies of war of the past and present, for deeds of ideal heroism, self-sacrifice, and devotion to duty. It seems like "carrying coals to Newcastle" to speak in an audience like this of what the railroads have done for all countries—for our own country especially, and more particularly for the Pacific slope. It was not so long ago when I read of the beginning of construction of the Trans-Siberian Railway which now unites the Atlantic and the Pacific on the other continent. The Cape to Cairo Railway, too, has passed through the stages of its preliminary surveys

and partial construction. What centuries of fighting could not accomplish, these two roads will in time accomplish. The light of civilization will be spread on the Dark Continent, and its strong rays will burst the fetters and open the prison doors of suffering men and women in Russia. Read the interesting description of what has already been done in the Sahara, in one of the recent numbers of *Scribner's Magazine.* The men who, like M. Rolland, have made the desert to "blossom as the rose", are they likely to see no further than the mere short span of the present? Does not their imagination people these stretches of land with busy towns and bustling marts, with peaceful villages and happy homes? Is it destructive of ideals to have contributed, if ever so slightly, to the realization of such a dream? True, the railroad has penetrated into the haunts and sacred preserves of our romantic friends—the wonderful majesty of the Alps, the somber beauty of the Black Forest, the towering ruggedness and inspiring beauty of our own Rockies, have become the possession of the many instead of remaining the privilege of the few. Nature has become vulgarized complains the romanticist, because for the many whose faith has been strengthened, nay created, as they view the wonders of Nature spread out before them, there may be shown a few whose souls are not attuned in sympathy with their surroundings—a few who in the midst of such surroundings cling to what is "of the earth earthy". Dr. Waldstein aptly says, in an admirable article on the works of Ruskin:

"There can be no doubt that our enjoyment must be impaired by the reduction of what stimulates our higher ambitions to a common place; but we must willingly make this sacrifice when we consider the great gain accruing to hundreds or thousands, where before it but reached units."

Who, then, is destructive of idealism? The man whose works are a means, if but a humble one, of bringing his fellow beings into a direct contact with the wonders of creation; or he who, enveloped in the mantle of exclusiveness, bemoans this defiling contact?

Now let us turn, if but briefly, to bridge engineering. Select from the many wonderful structures of the present century but one. To some a bridge is merely a means of safely transferring men and goods from one side of a valley, or one bank of a river, to the other. It may stand for much more to others. Take the famous East River Bridge.

Thousands throng it daily, on foot and by rail. Bright and cheerful, some cross in the morning but to return despondent and cheerless at night. Hopes deferred—hopes realized—obstacles overcome—overcome by obstacles: and so the stream of humanity flows across it day by day. Is that bridge merely so many pounds of iron, steel, and wood, suspended in mid air according to the laws of the strength of materials? Is that bridge but the embodiment of cold mathematical calculations? Surely it meant more to the noble engineer whose life was sacrificed in the building, to the devoted wife who added her patient hours of toil to those of her husband that he might not die before he had finished his great task. Was that a life devoid of ideals? Was that a work destructive of idealism?

What bridge engineer has not been touched by the romance of his Profession? What engineer has not been thrilled by Kipling's story of the "Bridge Builders" where that masterhand has voiced, in prose which is almost poetry, the thoughts that came to the engineer as he viewed his work:

"It was a long, long reverie, and it covered storm, sudden freshets, death in every manner and shape, violent and awful rage against red tape half frenzying a mind that knows it should be busy on other things; drought, sanitation, finance; birth, wedding, burial, and riot in the village of twenty warring castes; argument, expostulation, persuasion, and the blank despair that a man goes to bed upon, thankful that his rifle is all in pieces in the gun-case. Behind everything rose the black frame of the Kashi bridge—plate by plate, girder by girder, span by span—and each piece of it recalled Hitchcock, the all-round man, who had stood by his chief without failing him from the very first to this last."

If I desire to show that in still another branch of engineering—the one of river and harbor improvement—there is nothing destructive of idealism, I need not go far for an illustrious example. What James B. Eads has done for the people living in the Mississippi Valley in opening up the mouth of the Father of Waters, stamps him as one of the benefactors of mankind. For years he labored and fought, removed mountains of obstacles, overcame prejudice, malice and ignorance. Mr. Corthell, the staunch friend and principal assistant of Captain Eads on this momentous work, quotes him as saying:

"I therefore undertake the work with a faith based upon the ever constant ordinances of God himself; and so certainly as He will spare

my life and faculties for two years more, I will give to the Mississippi, through His Grace and by the application of His laws, a deep, open, safe, and permanent outlet to the seas."

Tireless energy, self-sacrifice, devotion to the welfare of mankind, make up a life as noble as any that was ever lived. Nor was this man bereft of ideals, who, when told, in a foreign country, that he was dying and could live but little longer, said: "I cannot die; I have not finished my work!"

Need I add to this a reference to that stupendous piece of engineering, which looms large in the memories of us all to-day. General Goethals, his predecessors and associates, have given the world an example of the earnestness, efficiency, and devotion which animates the members of our Profession. In the article previously quoted, General Chittenden says:

"As an example of national heroism—the making of a great sacrifice to accomplish a worthy purpose—it may rank with the most righteous wars."

May I be forgiven for mentioning by name one who contributed so much to the success of this enterprise, and whose name now is closely linked with that part of the work for which he sacrificed his life. Of him his chief has said:

"Colonel Gaillard was a great engineer and unflinching worker and a true gentleman. Gaillard Cut is a worthy monument to his name."

It is characteristic of George Eliot, that, with her marvelous insight into human nature, with that clear understanding which comes only of broad sympathies, she should have drawn that humble but lovely character of Caleb Garth—a type of the technical idealist. Do you recall the passage in which she says:

"Caleb Garth often shook his head in meditation on the value, the indispensable might of that myriad-headed, myriad-handed labor, by which the social body is fed, clothed and housed. It had laid hold of his imagination in boyhood. The echoes of the great hammer where roof or keel were amaking, the signal shouts of the workmen, the roar of the furnace, the thunder and plash of the engine, were a sublime music to him; the felling and the lading of timber, and the huge trunk vibrating star-like in the distance along the highway, the crane at work on the wharf, the piled-up produce in warehouses, the precision and variety of muscular effort wherever exact work had to

be turned out—all these sights of his youth had acted upon him as poetry, without the aid of the poets, had made a philosophy for him without the aid of philosophers, a religion without the aid of theology."

I think enough has been brought before you now to show clearly that engineering is not destructive of idealism. That much refuted, there still remains the charge that engineering is destructive of, or at least in part responsible for the decay of, art. I propose to show that this statement also is false. Artist and romanticist appear as accusers. Again they point backward and say: "See what the past has created; what have we that can be placed by its side?" Their eyes are blinded to the changed condition of things. They lack the sympathetic understanding of the complex problems of the life of to-day, and the materialized solution of these problems does not appeal to their idea of the beautiful. For the intelligent enjoyment, and more particularly for the criticism, of any creation, there is needed at least a fair knowledge of the underlying principles of construction, be that a work of symphony, a poem, or a bridge. It is true that a symphony or a poem appeals much more readily to a large audience than does a bridge or a complicated piece of machinery; yet both the latter may be as much works of art as the former, a higher degree of development of the intellect being needed, however, to see and feel their beauty. Every engineering structure is the materialized idea of its function. This first step in its construction gives us the core, the mere form, if you will; but, as I believe I have amply shown, the ideas underlying engineering works are often ideal ones, and the works themselves, therefore, can be idealized. When this is done, the engineering structure becomes a work of art. Its form may not at once strike us as beautiful, and keen discipline may have to be ours before we can see its beauties. But is that reason sufficient to condemn it? Ask the admirers of Browning, Whitman, or Wagner. It would seem, therefore, as though our idea of the beautiful is not a fixed one. In all ages that which most truthfully and characteristically embodied in itself the representation of the life and the ideas of those times was deemed a work of art. He is the artist who expresses most faithfully what we think and feel. If such representation has not been had in our century, it is not for lack of new ideas and materials furnished by science pure and applied, but for the lack of adequate assimilating power on the part of the would-be apostles of the beautiful. The cry

against machine-made ornament, machine-made reproductions of works of art, is justified, if it is directed merely against the untruths that often accompany such reproductions—if it is directed against the attempt to make material seem other than it is. But the cry becomes senseless when directed against those processes which place good, honest reproductions of beautiful works in the possession of the many, where once they were but the property of the few. Art will not become vulgarized if a good copy of the Sistine Madonna is found in every household, or a copy of the statue of the Venus of Milo in every town. One's love of the beautiful grows by being surrounded by beautiful things, and need there be any further justification than this for the plea of putting the imprint of beauty on the surroundings of our daily lives? Yet there are those who deny the claim to engineering structures. Art romanticists deny the possibility; strict utilitarians, the desirability; capitalists, the rentability. The possibility has been shown; witness the marvelous palaces from which radiate the myriad lines that bind us together in common activity, common enjoyment. Structures that mark an epoch, not worthy of being beautiful! Is it true, as Ruskin says: "That there never was more flagrant nor impertinent folly than the smallest portion of ornament concerned with railways or near them"? Is criticism such as this likely to prove as beneficial to the creation of works of art as is the work of men who are fully alive to the needs of the present, in sympathy with humanity of to-day, and desirous of giving that fact its noblest expression in their works?

Turn now, briefly, to the desirability of beautifying engineering structures. Our opponents become more dangerous because they number on their side some of the members of the Engineering Profession. All art is luxury, they hold, and unworthy of thinking man; hence engineering structures, which are largely the product of thought based on mathematical unalterable conclusions, should be left in all the nakedness of the constructive form. The beauty is there, they say, if you can but see it. And they are right in part, but, as Professor Lucae well says, "It is not a finished art form you are dealing with any more than is the human body with its exposed muscles and ligatures." The idealizing of the materialized idea of the function of the structure, therefore, surely seems desirable.

And right here it is encouraging to note that this desirability was recognized by our own Government in connection with the structures of the Panama Canal, when it sent the sculptor, Daniel C. French, and the landscape architect, Frederick Law Olmsted, to report on the artistic character of the Panama Canal structures, and to make suggestions for such improvements as to them seemed desirable.

"The canal itself and all the structures connected with it impress one with a sense of their having been built with a view strictly to their utility. There is an entire absence of ornament and no evidence that the aesthetic has been considered except in a few cases as a secondary consideration. Because of this very fact there is little to find fault with from the artist's point of view. The canal, like the Pyramids or some imposing object in natural scenery, is impressive from its scale and simplicity and directness. One feels that anything done merely for the purpose of beautifying it would not only fail to accomplish the purpose, but would be an impertinence."

Thus spoke the true artists.

Is it profitable to beautify engineering structures? Here we stand before a momentous question. If the answer be given by the engineer, or by one who holds that the status of a people is determined not merely by the accumulated wealth of the Nation, the quantity of goods produced, and of articles manufactured, then it will be in the positive, ten times over; but if the man of low ideals and mercenary motives gives answer, it is likely to be an emphatic "No". This answer has been given too often in our own country, and the blame for the deep scars in the face of Nature, the ugly dams and rugged cuts, must not be laid on the shoulders of the engineer, who fain would heal with loving hand, and protecting sward the wounds he has struck. Where broad-minded liberality and far-seeing policy govern the construction of engineering works, as is the case in countries older than our own, these works stand as worthy art products of the spirit of the times symbolical of the best and highest in the life of to-day.

Science and its applied form, Engineering, therefore, have not been destructive of idealism, for, in the words of another: "When the period of history we now call modern will be rounded to completeness, all the highest and most sacred human ideals will not be lost or dimmed, but will become nearer and more real," and science has not been destructive of art or beauty. As Emerson says:

"Beauty will not come at the call of a legislature, nor will it repeat in England or America its history in Greece. It will come, as always, unannounced, and spring up between the feet of brave and earnest men. It is in vain that we look for genius to reiterate its marvels in the old arts; it is its instinct to find beauty and holiness in new and necessary facts, in the field and road-side, in the shop and mill. Proceeding from a religious heart, it will raise to a divine use the railroad, the insurance office, the joint-stock company; our law, our primary assemblies, our commerce, the galvanic battery, the electric jar, the prism, and the chemist's retort; in which we seek now only an economic use. Is it not the selfish and even cruel aspect which belongs to our great mechanical works, to mills, railways, and machinery, the effect of the mercenary impulses which these works obey? When its errands are noble and adequate, a steamboat bridging the Atlantic between Old and New England and arriving at its ports with the punctuality of a planet, is a step of man into harmony with nature. The boat at St. Petersburg, which plies along the Lena by magnetism, needs little to make it sublime. When science is learned in love, and its powers are wielded by love, they will appear the supplements and continuations of the material creation."

MEMOIRS OF DECEASED MEMBERS.

EDWARD COOK BURNS, M. Am. Soc. C. E.*

Died October 29th, 1914.

Edward Cook Burns, the son of Barclay J. and Isabella Cook Burns, was born on January 30th, 1845, near the corner of Chambers Street and Broadway, in New York City, where his father was engaged in publishing and newspaper work. When the boy was three years old, his mother died, and he was taken to live with his grandmother at Jamestown, N. Y.

He received his early education in private schools at Jamestown. and was afterward sent to the Patterson Preparatory School in Detroit. Mich., where he was prepared for the Civil Engineering School of the University of Michigan from which he was graduated with the Class of 1868. Among his close friends and associates at the University were G. H. Benzenberg and the late Alfred Noble, Past-Presidents, Am. Soc. C. E., and Charles F. Brush, the inventor of modern electric lighting.

Mr. Burns had been employed during his summer vacations, from 1865 to 1867, as Assistant with the United States Lake Survey. In August, 1868, he was appointed Levelman on the Rockford, Rock Island and St. Louis Railway, remaining in that position until January, 1869, when he was made Division Engineer. From May to December, 1870, he was employed as Assistant Engineer on the Central Railroad of Iowa, and from April to July, 1871, as Division Engineer on the Burlington, Cedar Rapids and Minnesota Railroad.

Mr. Burns then returned to the East as Engineer in Charge of the extension of the Pittsburgh, Bessemer and Lake Erie Railroad until May, 1872, when he was appointed Assistant Engineer in charge of the construction of the Buffalo and Southwestern Railroad, from Gowanda to Jamestown, N. Y., under General Robert Ewing. From April, 1873, to August, 1874, Mr. Burns served as Acting Superintendent of the Jamestown Gas Light Company. In August, 1874, he returned to the Buffalo and Jamestown Railroad as Assistant Engineer, which position he retained until June, 1875.

From 1877 to 1885, he was employed as United States Assistant Engineer on river and harbor improvements at various places, including such important works as the construction of the Sault Ste. Marie Canal, the improvement of the St. Clair Flats Canal, the dredging of the Livingston Channel, etc., etc.

In 1885, Mr. Burns returned to Jamestown, N. Y., where, until his death on October 29th, 1914, after a year's illness, he was engaged

* Memoir prepared by the Secretary from information on file at the Society House.

in private practice as a Consulting Engineer. He was connected with many of the principal construction enterprises in Jamestown and the surrounding country, having served as Engineer of the Board of Public Works from 1894 to 1898. He was also one of the Conewango Swamp Drainage Commissioners and had charge of the construction of its line for the Chautauqua Traction Company. He was always in demand throughout the vicinity as an expert in the design and construction of highway and railroad bridges, and had planned and supervised the construction of the old and new Boatlanding Bridges and also the new Winsor Bridge, the latter two having been his last important work before ill-health compelled him to give up active practice.

On October 3d, 1872, Mr. Burns was married to Mary Graham, daughter of Major Thompson Graham, of Mercer, Pa., who, with one daughter, survives him.

Mr. Burns was a man of strong character and high ideals, who gave to his work his best thought and action. His strict integrity is shown in the character of his work, which was never slighted or cheapened to accomplish a temporary effect. He had a well-stored mind and, being fond of books and study, kept up with the advances and improvements of his profession. He was always considerate and courteous to every one, and will long be remembered by his friends and those who were closest to him for his pleasing personality, his kindly spirit, and his desire to add to the pleasures rather than the burdens of those with whom he was associated.

He was a member of the Delta Kappa Epsilon Fraternity and a Thirty-second Degree Mason.

Mr. Burns was elected a Member of the American Society of Civil Engineers on July 5th, 1882.

WILLIAM ROBERTS ECKART, M. Am. Soc. C. E.*

Died December 8th, 1914.

William Roberts Eckart was born on June 17th, 1841, at Chillicothe, Ohio. His father, William R. Eckart, was a merchant of prominence, who, at one time, had large shipping interests on the Lakes and, later, a managing interest in the Putnam Flour Mills, at Zanesville, Ohio. His mother was a member of the Carlisle family, pioneers in the settlement of Ohio.

After attending the public schools of Chillicothe and Cleveland, Mr. Eckart, having determined to take up the Profession of Civil Engineering, took a special course in mathematics at the St. Clair Street Academy, in Cleveland. Accepting the opportunity of serving an apprenticeship in the works of Griffith, Ebert, and Wedge, a firm of good reputation for general mill and steamboat work, he had the good fortune to come under the observation and to obtain the friendly interest of the junior partner, Mr. Wedge, who had had exceptional opportunities, in the shops of Sir Joseph Whitworth, in the manufacture of machine tools requiring extreme accuracy of construction. The effect of Mr. Wedge's insistence that "the best" was to be attained, was not lost on Mr. Eckart, but supplemented his natural ability and became apparent in all his work.

It was natural that his early opportunity to work on steamboat construction and to participate in trial trips, awoke in Mr. Eckart a desire to enter the Government service as a Naval Engineer. The application was made, and in the examinations of June, 1861, he received the highest rating of all applicants of that date.

On July 30th, 1861, he was appointed Third Assistant Engineer and ordered to join the fleet of naval vessels on the Pacific Coast. He arrived in San Francisco in the latter part of 1861, and soon became well known among, and formed friendships with, California's foremost engineers, including Paul Torqua, Joseph Moore, Irving M. Scott, Wallace Hanscom, Huttner, Specht, and many others.

In 1864, after resigning from the Navy on account of ill health, Mr. Eckart entered the employ of H. J. Booth and Company, at San Francisco, under Mr. Irving M. Scott who was Chief Draftsman, and was engaged principally on the design of mining machinery and steamboat work. As Chief Draftsman with the same firm, Mr. Eckart designed the first locomotive built in California, which was given its trial trip from San Francisco to San José on August 30th, 1865.

* Memoir prepared by C. E. Grunsky, A. M. Hunt, and Hermann F. A. Schussler, Members, Am. Soc. C. E., a Committee of the San Francisco Association of Members of the American Society of Civil Engineers.

In 1869, Mr. Eckart, then holding a license as First-Class Chief Engineer in the Merchant Service, accepted appointment as Draftsman in the Steam Engineering Department of the Mare Island Navy Yard, where, later, he was Foreman Machinist and Superintendent of Steam Machinery. A report by Mr. B. F. Isherwood and Mr. Eckart to the Secretary of the Navy on the relative efficiency of different propellers,* gives an account of the steam machinery, propellers, and dynamometers used in making the necessary experiments.

In 1871, Mr. Eckart left the Navy Yard, and entered into partnership with Prescott, Scheidel and Company, at Marysville, the firm name being afterward changed to Booth and Eckart. This firm specialized in hydraulic, milling, and mining machinery, and by reason of its favorable location did a prosperous business.

In 1876, the firm of Prescott, Scott, and Company, which succeeded that of H. J. Booth and Company, took some large contracts for pumping machinery for the Comstock Mines, and Mr. Eckart was recalled to San Francisco to assist in its design and to superintend its construction. By this time quite a rivalry had sprung up between Prescott, Scott, and Company (The Union Iron Works), and the Risdon Iron Works, in producing designs of machinery to handle the water and ores at Virginia City.

At this date the Sutro Tunnel was "in" 15 500 ft., had 5 000 ft. more to go, and would strike the Comstock Lode just below the 1 600-ft. level. The mine owners were already realizing that nothing but the heaviest and best designed machinery would meet the requirements on all the workings below the tunnel and, therefore, designs for pumps and hoists of large capacity to reach 4 000 ft. below the surface were decided on and called for. Not only the great depth, but also the ventilation and sinking of shafts in virgin ground abounding in large but unknown quantities of hot water to be met and overcome, presented problems unparalleled in engineering experience in any part of the world, and Mr. Eckart had a large share in dealing with and overcoming these conditions.

Having acted as Consulting Engineer for Sutro in sinking four shafts on the line of the tunnel, the investigations then made helped him materially when, in 1876, the orders came for the large pumps and hoists. He spent months in Virginia City experimenting, taking "cards" from pumps where the water would "boil an egg" and the vapors "air-bound the pumps", and where the expansion and contraction strains due to great changes in temperature often wrecked the heaviest castings. The knowledge thus gained was of great advantage to the firm he represented.

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* "On Marine Propulsion." *Transactions*, Inst. of Naval Architects of London, 1872, Vol. XIII, p. 315.

About this time, Mr. Eckart moved to Virginia City and became Consulting Engineer to the "Bonanza Firm", consisting of Messrs. J. W. Mackay, I. C. Flood, J. J. O'Brien, and James G. Fair, which owned or controlled nearly all the "North End" mines. During this time, he was Manager of the Fulton Foundry, at Virginia City. In 1878, he was also appointed United States Deputy Mineral Surveyor for the State of Nevada. During the following two years, a great part of his time was occupied in the underground workings of the Virginia City and Gold Hill Mines, investigating, planning, repairing, and improving the pumps and machinery, as the mines grew deeper and the water kept increasing. While still a resident of Virginia City, he, in connection with Mr. W. I. Salkeld, a noted millwright of the time, designed and built the Bulwer Standard Mill, at Bodie, which, at that time, was one of the largest pan mills for working ore.

During the early part of 1880, Mr. Eckart was appointed a member of the U. S. Geological Survey under Mr. Clarence King, and was given charge of investigating and reporting on "The Mechanical Appliances of the Comstock Lode." On this work, which was practically a labor of love, he spent nearly two years collecting data, testing pumps, engines, and hoists, and making drawings for the Government of all the machinery on the Comstock. The finest instruments procurable in the United States and Europe were used in the various investigations of efficiency. Hydraulic indicators tested and calibrated for from 500 to 5 500 lb. per sq. in. were used in testing the hydraulic pumps designed for the Cholar, Norcross, and Savage Shafts. Chronographs, to measure and record the velocities of engines and pump-rods, accurate to $\frac{1}{500}$ sec., were used on the surface and at a depth of 2 500 ft. The use of this instrument enabled results to be obtained which became of great value; diagrams taken from the heavy rods and pumps on the lower levels gave the clew to the location of the strains which had been so destructive in breaking rods and balance bobs in the past. The velocity curves of the rods and pumps revealed the fact, not theretofore known, and only in recent years properly understood, that the elasticity of long rods could give rise to free vibrations which at times were superposed on the force vibrations and accelerations due to the engines and pumps, so that maximum vibrations and strains resulted at certain parts of the rods, which exceeded the elastic limit of the timber of which the rods were constructed. Changes were thus determined in the location of balance bobs and weights, which increased the efficiency of the pumps and checked the destruction of the rods and bobs. Some of these velocity diagrams, as well as illustrations of the largest pump engines and the hydraulic pumps, were published by Professor A. Riedler, of Berlin, while acting as Commissioner of Mines to the German Government in 1893.

In 1881, Mr. Eckart removed to San Francisco and opened offices there as a Consulting and Constructing Engineer and, during the following eight or ten years, some of the largest and most important mining plants were designed and constructed under his supervision. The pumping engine for the Ontario Mine, with perhaps the largest Cornish pump for deep mining ever built in the United States, was constructed from his designs during this period. The pumps were 20 in. in diameter, with 10-ft. stroke; two pumps at each station operated from one pump rod, 2 000 ft. long. In 1881, he began, for Haggin and Tevis, plans for all the Anaconda Copper Works, Hoists, and Reduction Works, and during the following seven years he designed all the mining work and mills of this firm. The Anaconda Reduction Works, in Montana, which were started as a silver mill with a capacity of less than 350 tons per day, were increased in size and changed by additions until, in 1888, they were capable of working 3 000 tons per day. He designed and carried out much other work for the same firm.

In 1883, the Union Iron Works was changed to an incorporated company, and Mr. Eckart was retained as Consulting Engineer in matters pertaining to the propelling power of the Government vessels built by that Company. He was present at and assisted in conducting nearly all the preliminary and Government trials of these vessels.

In 1899, Mr. Eckart was appointed Consulting Engineer to the Standard Electric Company, and afterward became Engineer in charge of construction as well. He continued to act as Consulting Engineer after the completion of the plant, and after the Pacific Gas and Electric Company acquired his property as a part of its great power system, he was retained in the same capacity by the latter company, in connection with the hydro-electric branch of the work.

In 1907, the Snow Mountain Water and Power Company engaged Mr. Eckart as Consulting and Constructing Engineer, and he was connected with that company until he retired from active business at the close of 1913. He continued in the employ of the Pacific Gas and Electric Company, however, to the day of his death, which occurred on December 8th, 1914, at the residence of his son, Mr. W. R. Eckart, Jr., in Palo Alto, Cal. He is survived by his widow, Harriet Louise Eckart, and four children, W. R. Eckart, Jr., of the Leland Stanford, Jr., University, Charles F. Eckart, of Olaa, Hawaii, Nelson A. Eckart, M. Am. Soc. C. E., of San Francisco, and Mrs. C. E. Hume, of Piedmont, Cal.

Mr. Eckart had unusual talent as an experimenter and investigator. His methods were thorough, painstaking, and accurate, and his records of tests and experiments could always be regarded as absolutely trustworthy. Most of the special instruments and devices

needed in his various investigations were designed and made by himself. Among such instruments was a "Chronograph for Engineering Purposes, with the Hipp Escapement", described under that title by him in a paper* presented before the American Society of Mechanical Engineers, in 1882. In the discussion of that paper, the late Robert H. Thurston, M. Am. Soc. C. E., of Cornell University, speaks of Mr. Eckart as follows: "He is a very careful worker. In fact, I have not met in the profession a man who seemed better adapted to the application of fine measurements to experimentation on the steam-engine." This instrument was devised for use in the investigations made by Mr. Eckart on the Cornish pumps of the Comstock Lode, and was used subsequently in other investigations requiring accurate time measurements.

As a man and among his colleagues, Mr. Eckart had the admiration of all who knew him. He was a hard worker, always going to the bottom of his problems; and the mass of data which he had accumulated and the results of his ripe experience, were at the service of his colleagues whenever he was appealed to. His friendly help has been appreciated and will be remembered by all who have worked or conferred with him.

Mr. Eckart was honored by membership in the following Engineering Societies: American Society of Mechanical Engineers, April, 1882; Institution of Mechanical Engineers, London, January, 1878; Society of Naval Architects and Marine Engineers, May, 1893; American Society of Naval Engineers; and Associate Member of the Institute of Naval Architects, London.

Mr. Eckart was elected a Member of the American Society of Civil Engineers, on January 5th, 1881.

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* *Transactions*, Am. Soc. Mech. Engrs., Vol. 3, p. 184.

ALFRED NOBLE, Past-President, Am. Soc. C. E.*

DIED APRIL 19th, 1914.

Our America, America of the Twentieth Century, enlightened, progressive, prosperous beyond any other country of the world, would not be what it is to-day if it had not brought forth an army of engineers, an army small in numbers but mighty in accomplishment, always in the van of progress, making plain the highways of development in which the multitude marches on to attainments ever higher and higher.

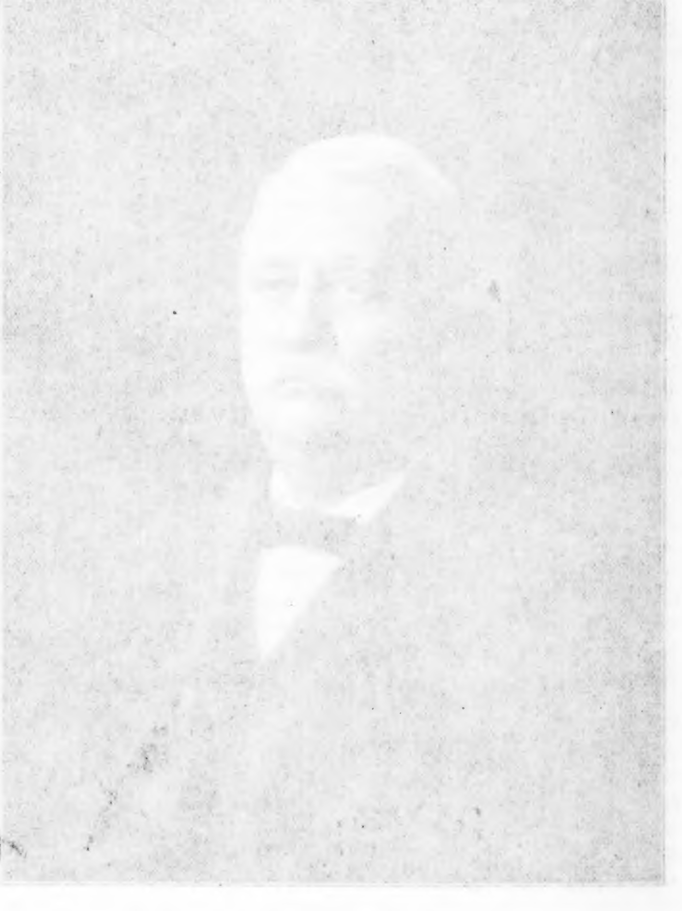
The work of this little army has never had just recognition, and these who have wrought the victories of peace tread the quiet paths of life and pass out of it, leaving behind works of enduring usefulness without having recorded upon such works in time-defying bronze or graven stone even their names. Not so with the army, which with trumpets blaring and flaunting banners marches into the "imminent deadly breach" and reaps a harvest of death and a halo of glory. There is scarcely a city or town in these United States which has not reared a monument to some military hero or some group of the rank and file whose support lifted the hero to the plane of public admiration.

The great and good men of our Profession are living their useful lives without a thirst for fame, doing the duty at hand faithfully and well, and passing resolutely on to the next which presents itself.

This was the history of Alfred Noble, who on April 19th, 1914. peacefully ended a useful life. A life so honorable, so kindly, so full of achievement that we delight to honor his memory, and would in some enduring way chronicle the deeds which have so eminently earned for him the love and admiration of his fellow-engineers. We, who thus delight to do him honor, are members jointly and severally of The American Society of Civil Engineers, The Western Society of Engineers. The Franklin Institute, The American Society of Mechanical Engineers, and The Institute of Consulting Engineers. Ours is a common loss, a community of feeling, and unitedly we record our appreciation of the man, the engineer, and the faithful friend. His were the simple homely virtues of truth, honesty, industry, and human kindness. Upon the solid foundation of sterling character he reared the superstructure of a splendid manhood, the story of which we would perpetuate, not only to do him honor, but

* Memoir prepared by a Sub-Committee (composed of Ralph Modjeski, Onward Bates, and Isham Randolph, Members, Am. Soc. C. E.) of a Joint Committee appointed by The American Society of Civil Engineers, The Western Society of Engineers, The American Society of Mechanical Engineers, The Franklin Institute, and The American Institute of Consulting Engineers.

ALFRED NOBLE



to hold up to the men who come after us a worthy example of a professional career to be emulated.

In an old family Bible it is recorded that on August 7th, 1844, a son was born to Charles Noble and Lovina Douw, his wife, at Livonia, Wayne County, Michigan. This son received the Christian name of Alfred. The child grew and waxed strong in body and in mind, and in those early and impressionable years no doubt the teachings of his mother helped to mould the character which commanded the respect and admiration of all those with whom he came in contact. She is described as being a "Woman of remarkable qualities, self-reliant. precise, austere, pious—New England type." Many of Alfred Noble's best traits were due to her training.

Our land can never pay its debt to the Christian mothers, who have gone to their rest, and its hope must still hang upon Christian mothers living and yet to come.

The concise chronicle before us goes on to say of his "early life and education":

"Spent on his father's farm until the age of eighteen. Helped his father and brothers in the arduous work of clearing, draining, and cultivating land in the new Michigan Country. Meanwhile, obtained education at the district school, beginning attendance when four years old. His three brothers all died young. When old enough to ride a horse he attended the Union School, at Plymouth, Mich., where most of his early education was gained. Of studious disposition, learning rapidly and thoroughly, according to his old Teacher, Dr. Frisbee."

This is all we have from the family record, to which tradition contributes nothing, and the only side light we have is from the pen of Judge Edgar O. Durfee, of Wayne County, Michigan. He writes (May 27th, 1914):

"I have known Alfred Noble as long as I have known anybody. His farm home was about a mile from the farm on which I was brought up, and I saw him very often. Some of the time we attended the same district school, and the winter of 1861 attended the graded school in Plymouth where we were in the same class in higher algebra. From his earliest school days he always excelled in all of his studies. He was very studious, and as a boy was the same as a man, always truthful and always lived up not only to the letter but to the spirit of his promises."

Scant as is this record of the first eighteen years of his life, it is sufficient to show "the boy as the father of the man".

We have now reached the war period, and the record says:

"On August 9th, 1862, two days after his eighteenth birthday, he enlisted in Company C, 24th Michigan Volunteers. Before doing

so, however, he obtained the consent of his mother, who opposed his going, but who withdrew her opposition on his declaration that he felt it his duty to enlist."

Here let us interpolate from Judge Durfee's letter:

"One little thing illustrates his character in that regard [loyalty to truth]. He was eighteen years old on the seventh day of August, 1862, on which day he enlisted in Company C, Twenty-fourth Michigan Infantry. His mother was thoroughly imbued with the idea that card playing was the greatest vice in the army—although we of the rank and file learned that the games played there were only euchre and old sledge. She asked him not to play cards, and he promised her he would not play cards while in the army. He lived up to this promise strictly, although he watched the boys play and learned more of cards than most of them, and as much as any of them."

The Judge gives his estimate of Mr. Noble thus:

"I think he was the best boy and man I ever knew, taking him all in all. He was very quiet, not given to boasting, was a warm friend, and had as fine a sense of humor as any person I ever knew. I am sure that everybody who came in contact with him as a boy and man was his friend."

Once more we quote from the scant outlines afforded by the record:

"The regiment (24th Michigan Infantry) was raised principally in Wayne County, following Lincoln's call in June for 300 000 volunteers. It was one of the first regiments composing the 'Iron Brigade', so called by General Hooker for its behavior at the battle of South Mountain. The regiment, being drilled for a time, did not participate in that battle (South Mountain), but arrived at the front in time to get into action at Fredericksburg in December. From that time until February, 1865, Alfred Noble took part in all of the principal engagements of the Army of the Potomac. At Gettysburg his brigade bore the brunt of the first day's fighting, and in particular his regiment lost 80% of its number. Although compelled to retire because of greatly superior numbers, the resistance of the 'Iron Brigade' on that occasion is generally credited with saving the Union position and making possible the final victory on the third day, on which the result of the entire war hinged. Alfred Noble was never wounded during the war, but at one time was very ill in the hospital, and considered that he owed his recovery to the ministrations of the Sisters of Charity. He was mustered out of service with his regiment in June, 1865, with the rank of sergeant, having acted as orderly at corps headquarters for a period of five months.

"He kept a diary during the war and throughout the rest of his life. Thus is told, in a statement of about 225 words, the story of his young soldier's life during nearly three years of marching, fighting, and enduring, a fraction of one word of each of these 1 000 days of stress, hardships, and danger. We long for a glimpse of the diary that he kept during the period when our country was fused in the 'melting pot' of war."

James H. Brace, M. Am. Soc. C. E., in his personal reminiscences of Mr. Noble, says:

"In the long twilight after supper, Mr. Noble could sometimes be induced to talk of his war experiences. He was very reluctant at all times to discuss this subject. He seemed to believe that it was every good citizen's duty to serve, then when the war was over, go about his regular business as though nothing had happened; that the country owed him nothing for his services, and that there was no good in keeping up the old spirit."

Eugene W. Stern, M. Am. Soc. C. E., says:

"As you know, Mr. Noble was very loath to speak of his war experiences, and it was during the course of the last three years of rather close acquaintanceship with him that from time to time I was able to glean a few incidents. It appeared to me that he wanted to forget the Civil War. The humorous side he was more inclined to dwell upon. He never wore a Grand Army button, to my knowledge. He told me the following incidents:

"Some years ago, when he was Resident Engineer on the Memphis Bridge, a man wearing a Grand Army button, claiming to have been a Colonel or Brigadier General during the War, came to see Mr. Noble about some matter or other in connection with some material he was selling. * * * 'He told me', said Mr. Noble, 'that he had been at Chancellorsville', and mentioned a certain incident which I knew did not agree with the facts, which I told him. He seemed rather astonished at my information on this point, and asked me how I knew, and I told him that I was there. He asked me to what Army Corps I belonged, and what rank I held. 'I am the last surviving member of it', I said. He seemed curious to know, and I replied, 'The great Corps of Privates'."

Mr. Stern says, further:

"On another occasion, in talking with him about the requisite qualities of a good soldier, he said, 'Ability to withstand hunger, fatigue, and hard marching, were very essential qualities, but to be a good runner was also often a very useful attribute'."

We know that, youth though he was, he served his country as a fighting man for three long years, and emerged from the ordeal of war uncontaminated by its demoralizing influences and conscious in his modest estimate of himself, that he was capable of service to his country and his kind in the more congenial paths of peace.

As we know, he chose for himself a career which he had to enter through the door of educational preparation. How he went about it is briefly told in the skeleton record of facts to which we must frequently refer.

"From July, 1865, to September, 1867, he held a clerical position in the War Department at Washington, Adjutant General's Office. During this time he saved his earnings and prepared himself, with the help of private tutors, to enter college, with such industry that

in the fall of 1867 he entered the University of Michigan as Sophomore in the class of '70. Among his classmates were Justice William R. Day and Judge Rufus H. Thayer, of Washington, D. C. He was a member of the Alpha Delta Phi fraternity, and became vice-president of his class in his Junior year. While an undergraduate, he was absent a year and a half in Government employ, acting as recorder with the United States Lake Survey, and kept up his studies at the same time, taking his degree of C. E. in 1870."

Of this period Mr. Justice Day, writing under date of November 3d, 1914, says:

"It was my privilege to be a classmate of his in the University of Michigan, where we graduated together in the class of 1870. I have met him from time to time since, and have known of the great career which he has had in his Profession, and am glad to know that it is the opinion of his associates that he was among the first engineers of this country.

"I well remember when Alfred Noble came to the University of Michigan, where he entered the Sophomore class in 1867. He was somewhat older than the rest of us, and, in my opinion, far more able than any of us. He had had three years' experience in the army, and those who knew him there said that he had been a faithful and valiant soldier. I do not think any of his classmates ever heard him speak of his army career. He probably regarded it as merely a part of his duty, and not a thing to be talked about."

We shall quote again from this letter in another connection.

As we leave this brief mention of his college career, we have no thought of derogating from the honor of any other graduate of that great seat of learning when we say that the proudest name upon its roster is that of Alfred Noble; and Ann Arbor can set before its students no higher professional inspiration than the story of Alfred Noble's life and work.

We have now reached the period when that life's work began in earnest, and the years that follow are full of action and achievement.

We turn again to the record, which reads:

"From June to September, 1870, he was engaged on harbor surveys on the eastern shore of Lake Michigan and the western shore of Lake Huron. In October, 1870, he was put in charge of the work at Sault Ste. Marie, Mich. In 1873 it was found necessary to build a new canal lock at the Sault and to dredge and straighten the channel of St. Marys River, and Alfred Noble was placed in charge of the work as U. S. Assistant Engineer under General Godfrey Weitzel, of the U. S. Engineer Corps. In this work, which occupied nine years, he practically designed and supervised the construction until completion of one of the present locks, known as the Weitzel Lock. The lock embodied new features that attracted the attention of engineers both at home and abroad. Most previous locks had been filled by admitting water through slides in the upper gates, and the water was released in the same way through slides in the lower gates."

It is appropriate here to quote from Mr. Joseph Ripley's letter under date of June 5th, 1914:

"The lock Mr. Noble built at the 'Soo' was named for Godfrey Weitzel * * *, and he always gave Mr. Noble full credit for his part in the work at the 'Soo'. (See Johnson's Encyclopedia, article on St. Marys Falls Canal, which was written by General Weitzel.) When the first boat, *The City of Cleveland,* was locked through to the Lake Superior level, the occasion was made quite an event, and about twenty engineer officers were present. Mr. Noble did not ride with the officers on the steamer, but stayed on the wall, watching locking operations. I heard Major (later General) Roberts, author of 'Roberts' Parliamentary Rules', congratulate General Weitzel on the completion of the greatest lock in the world, a work which would be a great personal honor and give renown to General Weitzel personally and, through him, be credited to the Engineer Corps and add much prestige to it. General Weitzel replied that 'Alfred Noble deserved all of the credit for designing and building the lock'. * * *

"When General Sherman made a tour of the Western forts, Mr. Noble was directed by General Weitzel to meet the party on arrival at the 'Soo' and to show them about the lock work. Mr. Noble delegated his assistant, Mr. Davock, to meet General Sherman while he (Mr. Noble) went up to the head of the canal and stayed there all day, so as not to put himself at all forward in the presence of so notable a man."

These are two instances of the modesty which clothed the man like a distinctive garb; but the last one raises the question, is modesty a justification for disobedience of orders? General Weitzel ordered Mr. Noble to meet General Sherman; that order was disobeyed. This, however, is the only instance of insubordination of this man who respected rightful authority and honored the law.

On May 31st, 1871, he married Miss Georgia Speechly, of Ann Arbor, Mich. One son survives this union, and truly Frederick C. Noble has a proud heritage in his father's name and fame. Of his married life we, who compile this memoir, have no record, nor is it essential or even appropriate that we should have; but we know that Alfred Noble maintained in the seclusion of his family life the same true and admirable character that is known to us in his career.

We will not (from now on) follow step by step the upward strides which he made, until he reached the highest distinction which could come to any man in the Engineering Profession, any further than to note the date and the character of each attainment in usefulness and honor; but we will make a part of our presentation letters from distinguished men whose association with him peculiarly fitted them to record their appreciation of his work.

Enters Railroad Work in West.—In August, 1882, the canal being practically finished, he resigned from the Government service to become Resident Engineer on the construction of a railroad bridge across

the Red River at Shreveport, La., the late L. G. F. Bouscaren, M. Am. Soc. C. E., Chief Engineer. In March, 1883, he resigned this position to accept a similar one on the Northern Pacific Railroad, then nearly completed as to track laying, on the construction of a bridge across Snake River, at Ainsworth, Washington Territory, the late General Adna Anderson, M. Am. Soc. C. E., Chief Engineer. In September he was put in charge also of the replacement of a timber bridge over Clark's Fork of the Columbia, near Belknap, Mont. Both bridges were completed about the middle of the following year. In September he was put in charge of the construction of foundations of a high trestle across Marent Gulch near Missoula, Mont.; and in October of the foundations of a bridge across St. Louis Bay at Duluth, Minn. The Marent Gulch Viaduct was completed in June, 1885, including superstructure and new foundations. The St. Louis Bay Bridge was completed in May, 1885, according to original plans, and the construction of an additional draw-span was started in July. From August to October of this year was spent at Trenton, N. J., inspecting the ironwork for the draw at the shops, and from October to the following January, 1886, he was supervising its erection. In February, 1886, he was in New York City, in the office of the late George S. Morison, M. Am. Soc. C. E. During March and April, he was inspecting bridge manufacture at Buffalo, and in May was inspecting iron at Pottsville, Pa. He then returned to New York in June. He visited Omaha Bridge in July, and then went to St. Paul for temporary duty with the Northern Pacific Railroad as Acting Principal Assistant Engineer. In September he went to Pittsburgh to inspect ironwork.

Washington Bridge.—In October, 1886, he resigned to accept an appointment as Resident Engineer of the bridge across the Harlem River at 181st Street, New York City, since known as the Washington Bridge. The late William R. Hutton, M. Am. Soc. C. E., was Chief Engineer.

Cairo Bridge.—In July, 1887, he resigned to accept an appointment as Resident Engineer of the Illinois Central Railroad bridge over the Ohio River at Cairo, Ill., the late George S. Morison and Elmer L. Corthell, Members, Am. Soc. C. E., Chief Engineers. This bridge was opened for traffic on October 29th, 1889.

Memphis Bridge.—In November, 1889, he assumed charge, as Resident Engineer, of the railroad bridge over the Mississippi River at Memphis, Tenn., the late George S. Morison, Chief Engineer. This bridge was opened for traffic in May, 1892.

Partnership with Mr. Morison.—On the completion of the Memphis Bridge, he entered a limited partnership with the late George S. Morison in Chicago, which lasted until April 30th, 1894. During this term he was Assistant Chief Engineer of the bridge across the Missis-

sippi at Alton, Ill., and of the bridges across the Missouri at Bellefontaine, Miss., and Leavenworth, Kans.

Enters Private Practice.—On the expiration of the partnership, in April, 1894, he established an office in Chicago and entered general practice as Consulting Engineer. During the first two years of his practice, he was connected with various constructions, including the regulating works of the Chicago Main Drainage Channel, a power canal at Sault Ste. Marie, the foundations of a bridge across the Harlem River, New York City, foundations of office buildings in the lower part of Manhattan Island, and a wharf at Tampico, Mexico.

In April, 1895, he was appointed by President Cleveland as a member of the Nicaragua Canal Board. The board visited Central America, examined both the Nicaragua and Panama Canal routes, returned to the United States, and completed its work in November, 1895.

In July, 1897, he was appointed by President McKinley a member of the United States Deep Waterway Commission, to make surveys and estimates of cost for a ship canal from the Great Lakes to deep water in the Hudson river.*

In June, 1899, he was appointed by President McKinley as a member of the Isthmian Canal Commission, which was charged with the determination of the best canal route across the Isthmus. The Commission visited Europe, examined the data relating to the Panama Canal collected in the office of the French Company, in Paris, and visited the Kiel, Amsterdam, and Manchester Ship Canals.

In the spring of 1898 he was appointed by William R. Day, then Assistant Secretary of State, as arbitrator in a dispute between a citizen of this country and the Government of San Domingo. He visited that Republic, returning to New York a few days before the declaration of war with Spain.

In the fall of 1900 he was appointed a member of an engineer board to advise the State Engineer of New York concerning the plans and estimates for a barge canal across that State.

In November, 1901, the city authorities of Galveston, Tex., appointed him as a member of a board of engineers to devise a plan for protecting the city and suburbs from future inundations. This board reported a plan involving the building of a solid concrete wall more than 3 miles in length and 17 ft. in height above mean low water, the raising of the city grade, and the making of an embankment adjacent to the wall, the whole to cost about three and a half millions of dollars.

In November, 1901, Mr. Noble formed a partnership with Ralph Modjeski, M. Am. Soc. C. E., for the purpose of engineering the Thebes Bridge over the Mississippi River at Thebes, Ill.; and the

* *Engineering Record*, April 25th, 1914, p. 466.

corporation representing the several interests, for which that bridge was to be built, engaged these gentlemen, thus associated jointly, to design and build that bridge. From that time until January, 1905, Mr. Noble, without neglecting any of the other great engineering works with which he was identified, devoted a great deal of time to this bridge, one of the most massive and imposing of the many now spanning the Father of Waters.*

From 1902 to 1909 Mr. Noble was Chief Engineer of the East River Division of the New York extension of the Pennsylvania Railroad, and was in entire charge of this most difficult piece of work, involving as it did a very accurate survey across Manhattan and the construction of the foundations of the Pennsylvania Station, of the land tunnels, and of the East River Tunnels.

In 1905 he was appointed by President Roosevelt a member of the International Board of Consulting Engineers for the Panama Canal. This board was commissioned to advise the President and the Congress of the United States concerning the type of the canal. Its work has passed into history, and the lock canal across the Isthmus is to-day a monument to the wise counsels of five Americans who, through a minority report, convinced the President and the Congress that a sea-level canal should not be considered. Of the membership of that board all but Alfred Noble survive, and each of the twelve survivors is ready to bear testimony to the splendid work which he contributed to the labors of the board and to recognize the potency of his influence in bringing about the results, of which, as a people, we are proud to-day.

Nor did his work in connection with the Panama Canal end with the life of that Board of Consulting Engineers, for his advice was later requisitioned in connection with vital problems in its construction, such as Gatun Dam and the lock foundations.

During the four years, from January, 1910, until the time of his death in April, 1914, Mr. Noble's work in private practice covered a broad field. He was called on for advice twice by the United States Government and twice by the Canadian Government, he was twice employed by the City of New York as consulting engineer, and acted as consulting engineer for various corporations, reporting on ten different water-power projects. All of the foregoing involved careful studies in what is commonly referred to as the theoretical side of engineering. An evidence of the breadth of his experience is the fact that during the same time he was employed on nine different occasions by contractors on large construction works to advise them in regard to the so-called purely practical questions involved in carrying out their works.

In addition to all of the above, he found time during this period to undertake, in a public-spirited desire to benefit the Profession, a

* Fuller details are given in the Appendix.

large number of gratuitous tasks, such as serving on various committees. The amount of time and labor which he gave to this work was very great, amounting to about one-third of his total time. This will be appreciated only by those who were associated with him.

In behalf of the United States Government, he went to Honolulu to examine and report upon the Pearl Harbor Dry Dock. He spent several months in making a study of this problem, in the very thorough manner which was one of his most notable characteristics, and submitted a voluminous report to the Secretary of the Navy.

After the completion of the plans for the New Welland Canal he was employed by the Canadian Government to report upon all the plans and the projects as a whole. His report on this work to the Minister of Railways and Canals was dated May 13th, 1913. He had, previous to this, advised the Canadian Government on the foundations for the new Quebec Bridge, after the fall of the old bridge and while the plans were being drawn up for the new structure just prior to the letting of the contracts.

He was engaged on New York City work from October, 1909, until his death. The city made use more particularly of his ability as an expert in tunnel matters, first, on the many miles of tunnel for the Catskill Aqueduct north of the city and the deep tunnels under the Boroughs of Manhattan and Brooklyn and the East River, and, secondly, on the subway tunnels, especially the four East River tunnels known as Routes 33 and 48.

The first of the water-power projects involved a study of the regulation of Lake Superior for the Michigan Northern Power Company. This problem covered four years of continuous work, and the report, filling three large volumes, is now filed with the International Waterways Commission. A surprisingly large proportion of this work was done by Mr. Noble personally; if he had a weakness, it was in this habit he had formed of doing possibly too much work himself.

He visited California twice to examine and report upon projects for the Big Meadows Dam for the Great Western Power Company, and gave a large part of his time, extending over a year, to the study of a power development on the Susquehanna River. He also made a study of an extension of the plant at Niagara Falls; a study of power possibilities on the St. Lawrence River; and a report on a plant at Grand Falls, New Brunswick.

Aside from the 160 million dollars, more or less, which will be the cost of the Catskill Aqueduct, and which he cannot be said to have passed upon as a whole, the value of the work referred to him for his judgment during the four years totals nearly 100 million dollars. This is mentioned only as giving some idea of the magnitude of the responsibilities which were placed upon him, and as an indication of the value placed upon his judgment.

It is almost past believing that all this work should have been performed by one man in 52 years; and that of those 52 years three were given to military service in time of war. Such a record attests the wonderful mental and physical power of the man, his steadfastness to duty, and his ability to endure the strain of such a life. There is enough in that record to have made several men great, had those activities been equally apportioned to them.

Now let us record the honors which he won so worthily and wore so modestly:

Honors Conferred.—

President, Western Society of Engineers, 1898.

President, American Society of Civil Engineers, 1903.

Honorary Member, Institution of Civil Engineers, Great Britain, 1911.

President, American Institute of Consulting Engineers, 1913.

Degree of LL.D., University of Michigan, 1895.

Degree of LL.D., University of Wisconsin, 1904.

Awarded John Fritz medal for "notable achievements as a civil engineer", 1910.

Awarded Elliott Cresson medal of Franklin Institute for "distinguished achievements in the field of engineering", 1912.

Other Distinctions.—

Member of Tau Beta Pi.

Chairman, Joint Conference on Uniform Methods of Tests and Standard Specifications for Cement, 1914.

Member of Special Committees of American Society of Civil Engineers, reporting on Uniform Methods of Tests of Cement, 1885 and 1912.

Chairman of Special Committee to Investigate Conditions of Employment of, and Compensation of, Civil Engineers, 1913.

Presented general report on "Dimensions to be assigned, in any given country, to canals of heavy traffic. Principles of operating. Dimensions and equipment of the locks" to the XIIth International Congress of Navigation, at Philadelphia, 1912.

Member, Board of Managers, American Society of Mechanical Engineers, 1912-14.

Vice-President, Engineering Section, American Association for the Advancement of Science, Annual Meeting, 1914.

(Died before he could serve; succeeded by F. W. Taylor.)

33d degree Mason.

Vice-President, Engineers' Club of New York.

Director, American Highway Association, 1912-13.

Membership.—

American Society of Civil Engineers.
American Society of Mechanical Engineers.
Canadian Society of Civil Engineers.
Western Society of Engineers.
Institution of Civil Engineers.
American Institute of Consulting Engineers.
Permanent International Association of Navigation Congresses.
American Highway Association.
Engineers' Club of New York.
University Club of New York.
Century Club of New York.
Chicago Club.
Chicago Engineers' Club.
Alpha Delta Phi Fraternity.
Various national and local scientific and economic organizations.

These all were his, but no word of his ever betrayed pride in their possession, although he prized every manifestation of the love, esteem, and honor with which his fellow-men regarded him.

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General Report (International Navigation Congress, 1912) on dimensions to be assigned, in any given country, to canals of heavy traffic. Principles of operating. Dimensions and equipment of locks.

Railway Tunnels of New York City, *Journal*, Franklin Inst., April, 1913.

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Marked Characteristics of the Man.

Modesty.—We have spoken of his modesty, and would further emphasize it by a few quotations:

"Not the least of his great virtues was inherent modesty, * * *. Not long after entering on his engineering work he was offered a professorship, under conditions which led him to look upon the offer with favor, but his friends of those days—of which I am proud to have been one—felt that Noble was destined to become an active participant in the great construction work of his profession, to a much greater extent than his natural modesty permitted him to admit, and fortunately our counsels prevailed."—(A. Mackenzie, Hon. M. Am. Soc. C. E., Maj.-Gen. (*Retired*), Corps of Engrs., U. S. A.)

"While Mr. Noble undoubtedly understood and knew of his marked ability, he never appeared to realize that he excelled or to assert it, but had a quiet, unassuming, reserved, and kindly personality which was most attractive."—(Joseph Ripley, M. Am. Soc. C. E.)

"He was probably one of the most modest men in the Profession, and never failed to accord to his chief all credit for the conception of the work and the principal administration of it."—(William H. Burr, M. Am. Soc. C. E.)

"In his case, the boy was father to the man. He was modest, kindly, industrious, and capable, as boy and man."—(Justice William R. Day.)

Industry.—

"He always came to the office first and usually left last. No matter how some of us tried to be on the work ahead of him, we always found Mr. Noble there, * * *. No work was too trivial or too irksome for him. Nothing was neglected or passed over."—(Ralph Modjeski, M. Am. Soc. C. E.)

"Throughout the whole laborious operations of the first Isthmian Canal Commission, Mr. Noble bore his full share from the beginning to the end, and his services aided much in giving to the report its high value."—(William H. Burr, M. Am. Soc. C. E.)

"His professional work upon this [Isthmian Canal] Commission was of a very high order. With untiring industry he mastered the details of every branch of the investigation, and then with sound judgment and judicial temperament he reached conclusions which could not be shaken. * * *.

"My subsequent association with Mr. Noble, aside from the Panama Canal, related particularly to the hydraulics of the Great Lakes, and confirmed me in the conviction that, for the solution of any engineering problem involving long and careful analysis, he had no superior."—(O. H. Ernst, M. Am. Soc. C. E., Brig.-Gen. (*Retired*), Corps of Engrs., U. S. A.)

"While at the 'Soo', Mr. Noble did the work of three or four expert engineers. He worked twelve to eighteen hours every day."—(Joseph Ripley, M. Am. Soc. C. E.)

"So frankly honest was he, that his whole life was an open book from the time he quietly entered upon his chosen profession at the Sault Ste. Marie Canal—while still a student—up to and through his career as a world-known master."—(A. Mackenzie, Hon. M. Am. Soc. C. E., Maj.-Gen. (*Retired*), Corps of Engrs., U. S. A.)

Ability to Handle Men.—

"His treatment of his subordinates was exceedingly kind without being lenient. Always ready to help with word of advice or to turn up his sleeves and join in the work if he saw he could help."—(Ralph Modjeski, M. Am. Soc. C. E.)

"He chose his assistants with care, and required a great deal of them, although not as much as he demanded of himself. When he had once given a man his confidence, he was entirely willing to leave to him the carrying out of his instructions, and such suggestions as he made were always conveyed in a kind, generous manner, which made it a delight to talk over any point with him."—(Henry Goldmark, M. Am. Soc. C. E.)

"When several hundred men were employed on the work [at the 'Soo'], he knew and called every one by name, and could tell the value of each man as a workman. He was always pleased to find any employee taking special interest in his work, and would cheerfully aid in furthering that interest by explanation, by teaching, or in other ways. * * * . He was a great and most successful leader. It was no wonder that all his employees were loyal to him, willing to give the uttermost possible to acceptably serve him."—(Joseph Ripley, M. Am. Soc. C. E.)

Cheerful Serenity and Kindly Humor.—

"With his great qualities and achievements, he had a gentle vein of humor that made him the most agreeable of companions."—(Mr. Justice William R. Day.)

"He was not only the experienced professional man, but most gracious and invariably kindly in his relations with every member of the Commission. He was patient in times of difficulty, and frequently lightened the troubles of many unwelcome conditions by bits of quiet humor in which he was wont to indulge."—(William H. Burr, M. Am. Soc. C. E.)

"During the trip there were some trying experiences from wind and weather, but throughout these, as well as during the sunshine, Mr. Noble displayed the same kindly good humor and thoughtful consideration for others that characterized all his relations with his fellowmen."—(James H. Brace, M. Am. Soc. C. E.)

"Throughout these expeditions [Nicaragua and Panama] Noble's equanimity never for a moment deserted him. His sweetness of disposition and generosity of temper endeared him to all."—(O. H. Ernst, M. Am. Soc. C. E., Brig.-Gen. (*Retired*), Corps of Engrs., U. S. A.)

Sense of Obligation.—

The largest obligation received loyal discharge from him, and the least was never forgotten nor neglected.

"Another characteristic incident: When Mr. Noble was going to Panama, he asked me to keep his club dues paid. 'For', he said, 'I should not like to be posted as delinquent, and again I should dislike *not* to be posted if I deserved it."—(Ralph Modjeski, M. Am. Soc. C. E.)

Generosity.—

"He was generous. By accident I have learned of several instances where he has contributed considerable sums of money regularly for one or more years where employees have been injured, or who had dependent families sorely in need of assistance."—(Joseph Ripley, M. Am. Soc. C. E.)

"He was most scrupulous and generous in money matters. Always ready and desirous to give more than he received, not only in money matters but in everything else."—(Ralph Modjeski, M. Am. Soc. C. E.)

"In addition to all of the above, he found time * * * to undertake, in a public-spirited desire to benefit the Profession, a large number of gratuitous tasks, such as serving on various committees. The amount of labor which he gave to this work was very great, amounting to about one-third of his total time. This will be appreciated only by those who were associated with him."—(S. H. Woodard, M. Am. Soc. C. E.)

"He was generous and kindly, and more considerate of others than he was of himself. * * *. I remember a little incident which occurred about a year ago, shortly before the change in the City's administration. He told me he could see that there was to be a strong cry for economy in all of the operations of the City, and that, whether it was advisable or not, strong pressure would be brought on our Board for a reduction of expenses, and so he suggested that, as he was the last of the three Consulting Engineers to be appointed he would retire, in order that the others might not be disturbed. Neither the members of the Board nor I would listen to such a proposition, as we believed that his counsel was too valuable to lose at a time when the work was to be put under test, and his services might be very necessary."—(J. Waldo Smith, M. Am. Soc. C. E., Chief Engineer, Board of Water Supply, New York City.)

Faithfulness.—Just one instance, characteristic of the man:

"In the death of Alfred Noble the American Highway Association has lost its greatest and most useful member. * * *. He attended the founders' meeting, and was there elected a member of the Executive Committee, on which he served to the time of his death. During the four years that he served on this committee he never missed a meeting."—(Logan Waller Page, M. Am. Soc. C. E.)

A greater than all has said: "He that is faithful in that which is least is faithful also in much." The law of faithfulness is not limited by magnitude; it obligates from the least to the greatest, from the greatest to the least.

SPONTANEOUS EXPRESSIONS OF SORROW AND OF SYMPATHY.

"The name of Alfred Noble will live in our memories, and in history, with those who possessed the finest qualities of heart and intellect."—(Charles S. Carter.)

"My relations with him for many years past had been very close, in connection with the New York tunnel work in which we were so closely associated. He was a man for whom every one entertained the highest respect, not only for his professional ability and talents, but for his many endearing personal qualities as well.

"The Profession has sustained a great loss in Mr. Noble's death, and I beg to assure you of the very deep sense of loss which I feel personally."—(Samuel Rea, M. Am. Soc. C. E.)

"I first knew him as the great engineer, but came to know him also as the biggest, broadest, and most human man with whom I ever came in contact."—(Paul G. Brown, M. Am. Soc. C. E.)

"Like every one else that knew Alfred Noble, I not only admired him as a man and as an engineer, but had for him a real deep affection as a friend, and I feel that I too have suffered a loss to-day."—(William Barclay Parsons, M. Am. Soc. C. E.)

"He stood for nothing but the straight, unvarnished truth, and I am sure there was not a man who knew him but felt he was the better for having known him and the better for following him."—(James Forgie, M. Am. Soc. C. E.)

"There has been nothing that I have felt more than his death; he was so good, kind, generous, always thinking of your comforts."—(George Kemp.)

"I, too, have keenly felt the loss of my great, good friend and chief, for the death of your father was a sad personal bereavement to me, the severing of an acquaintance of thirty-eight years.

"Alfred Noble was my ideal of a man, a grand character embodying the best traits of human intelligence and personality. He measured up to the perfect standard of a Chief Engineer, with full technical and practical ability, ready with right expedients, always successful, with never a failure, with unassuming modesty, with a living honesty of intent and deed, bright and spotless as sunlight, and an inborn gift of leadership which inspired loyalty to him and his work in every employee, however humble or important the position occupied might chance to be, and imbuing a spirit of service willing to go to the limit of uttermost endurance. I recall the tribute paid by a noted speaker in 1871 to the memory of a beloved teacher, and the words fittingly describe Mr. Noble's kind, gifted and forceful personality:

'Ah! one I saw and still can see,
As a picture dim he seemeth to me
By the hand of a Master painted.
Around the picture a halo clings,
And the face that memory backward brings
Is like the face of the sainted.
A richer wealth than the gold of fools,
A wiser wisdom than dwells in schools,

A nobler honor than place confers,
And a power a Prince might boast was his.
So deeply cultured in word and thought,
Each taste and talent so finely wrought,
So ran the purity of his life,
So sweet the harmony of its strife,
So grand the result of his being's task,
The world for ages was greatly blessed.'"

—(Joseph Ripley, M. Am. Soc. C. E.)

"Somehow, it seemed hard to think he was gone, and that it ought not to be. I was an admirer of him and attached to him in kindly feelings of long acquaintance. He was to me a remarkable man, whom I counted as one of the few really great men I have known; and I always rejoiced in learning of the deserved recognition of him by others. While he undoubtedly understood and knew of his marked ability, he never appeared to realize that he excelled, or to assert it; but had a quiet, unassuming, reserved and kindly personality which was most attractive to me. He was really a man that none knew but to love and none named but to praise. It is a gratification to remember that he won appreciation and distinction in his life work and did his life work grandly. To us who knew him so long, his going leaves a special sadness, and I can particularly understand the feeling of lonely sorrow and great loss of a close friend in your case. I deeply sympathize with you; and join with you in the sentiment that one of the grandest, and most useful, men of this country has gone."—(J. H. Steere.)

"His services to the City and State were permanent and lasting, and he will be greatly missed by the many friends who fully appreciated his high character."—(William R. Willcox.)

"It is very possible that you have never heard my name, but as it was a great privilege for me to know your father, I want you to know that I am one of a legion of men who owe to your father a debt of gratitude. The great simplicity, truth, honor, and ability which Mr. Noble stood for was not only an inspiration to me, but has done a great deal to establish and re-establish my faith in my fellow-men."—(Walter F. Dillingham.)

"He was one of those great men whose modesty, gentleness, and kindliness vested his greatness with a charm, and made all those who knew him love him as a man as strongly as they admired him as an engineer.

"His loss makes a huge gap in our ranks, as in our hearts, and it is hard to realize how it can ever be filled."—(R. S. Buck, M. Am. Soc. C. E.)

"It was a very great pleasure to me to have met your father, for I esteemed him very highly as a man and as an engineer."—(J. S. Langthorn, M. Am. Soc. C. E.)

"A great loss to the Engineering Profession and to the community."—(F. W. Carpenter.)

"Since chance has thrown us together, and I have had the pleasure of knowing him rather intimately, I have been struck with admiration

for his great abilities and his splendid character as a man."—(Josephus Daniels.)

"I am mourning the best of men and the best of friends."—(Ralph Modjeski, M. Am. Soc. C. E.)

"It is impossible to think of any one in the Profession who will be more sadly missed."—(Arthur S. Tuttle, M. Am. Soc. C. E.)

"I cannot express to you what I feel, in the loss of your noble father, and can only say now that his life has been an inspiration to all those who have had the good fortune to know him, as I have.

"Let me add that while his presence has gone, his personality will ever be a precious memory to us."—(Eugene W. Stern, M. Am. Soc. C. E.)

"I esteem it a great privilege to have known him and to have been, even to so small an extent as I personally have been, associated with the one man who in my opinion outranked all the other engineers in this country. His splendid character and honor have been a great influence for good in the Profession, and we will all miss that fine guiding spirit very greatly. * * * In his death there still remains to you and to us the memory of one of the finest men that ever lived, finishing his course in the full possession of all his powers and at the summit of his fame. What can any one of us desire for himself better than that."—(J. Vipond Davies, M. Am. Soc. C. E.)

"I am greatly shocked by your announcement of the death of Mr. Alfred Noble. I have found great satisfaction in the fact that during my term of office as President of the A. S. M. E. Mr. Noble sat at our Council Board. Meeting him there gave me the first opportunities I have had for gaining his acquaintance. I, of course, knew of his work as an engineer and of the fine reputation he has always sustained as a man; but as my acquaintance with him increased, and as I came under the influence of his personal charm, I appreciated as never before the significance of his presence at our meetings and, in a larger way, of his life among men."—(W. F. M. Goss.)

"Mr. Noble was not only one of our great engineers, but the highest type of man in every respect, and his quiet, lovable ways endeared him to all. He will be greatly missed, and it will be very difficult to fill the position he has occupied in the engineering world."—(Ambrose Swasey.)

"He was certainly one of the greatest men in our Profession, and his unassuming personality, coupled with his wonderful achievement, should make him a model for all of us to follow.

"I know of no single man who, it seems to me, will be a greater loss to our Profession."—(Fred. W. Taylor.)

"I am desired to inform you that at the first Meeting of the Council of this Institution after the death of our distinguished Honorary Member, Mr. Alfred Noble, the following Resolution of Condolence was passed:

"Resolved: That the Council record the deep regret with which they have learned the death of Mr. Alfred Noble, Honorary Member, who always evinced the warmest interest in the affairs of The Institu-

tion since he was elected a Member in 1901."—(J. H. T. Tudsbery, *Secretary,* Inst. C. E.)

"As a classmate of my father's at the University of Michigan, his name is familiar to me from my earliest recollection, and my personal acquaintance with him, dating from my first employment in Nicaragua sixteen years ago, was a source of great pleasure to me. I value greatly the opportunities that I have had to know him there and in Panama, and later in this city."—(Henry Welles Durham, M. Am. Soc. C. E.)

"I knew him well, and he was a splendid man. We will all miss him very much."—(Ralph Peters.)

"Although an entire stranger to you, I trust you will permit me to express my profound and sincere sorrow because of the death of your honored father, the late Alfred Noble, than whom I never knew a finer gentleman or better engineer. Truly we have lost our best.

"My acquaintance with him was but slight, * * * and although I have met him but seldom in recent years, I have cherished the memories of my slight association with him, and never failed whenever I saw his name to experience a thrill of pleasure as I recalled the genial face and pleasant smile of him who, above and beyond all his other rare qualities and attainments, was always four-square to every one."—(W. L. Smith.)

"I feel it myself a great deal. I had known him for over thirty years and feel more admiration and affection for him than I can tell."—(Edwin Duryea, Jr., M. Am. Soc. C. E.)

"Many of us, who had the privilege of association with your father sympathize with your sense of separation and loss. We loved and respected him as a man and a counsellor. * * * His fine, long record of usefulness has been and will be an inspiration to many."—(Alfred Douglas Flinn, M. Am. Soc. C. E.)

"I know of no one who had a higher regard for your father than I had, and I shall miss him greatly. He was a man who lived up to his name, and his loss is world-wide."—(Louis H. Barker.)

"I cannot refrain from telling you of the great admiration I had for your father and the high affectionate esteem I, and all others who knew him, had for him."—(Daniel E. Moran, M. Am. Soc. C. E.)

"Your father was great, not only in respect to his achievements, but also in that he commanded the love and honor of every one whose privilege it was to know him."—(Waldo C. Briggs, M. Am. Soc. C. E.)

"We have always considered Mr. Noble one of the big, strong, capable men in this country, and as such, he won the esteem and admiration of every one, including ourselves."—(Bradley Contracting Company.)

"I believe that the world had lost in your father its foremost civil engineer, and we all feel that we have lost a valued personal friend."—(S. P. Brown, M. Am. Soc. C. E.)

"I am very sorry to hear of the death of Alfred Noble. He was the dean of American engineers and has left a record of brilliant usefulness upon which it is inspiring to dwell. I had at one period much

official relationship with him and came to respect him most highly as a man and as an engineer. His professional advice in respect to the type of the Panama Canal and the security of the foundations of the Gatun Dam was followed by the Government and has been vindicated completely by the event."—(Wm. H. Taft.)

"I have for many years held Mr. Noble in high esteem, both as a man and as an engineer. The country is under great obligations to him for his wise and far-sighted course in relation to the Panama Canal. As a member of the International Board of Consulting Engineers, assembled by President Roosevelt in 1905, he threw the weight of his long experience and acknowledged engineering ability in favor of a lock as against a sea-level canal and wrote the report of the minority members of that body, in which the plan of the canal as constructed was outlined. As a member of a special commission of three sent by President Roosevelt to the Isthmus in 1907 to make a special investigation of the lock and dam sites, his signature to a report declaring the foundations safe and stable had great effect in reassuring public confidence."—(George W. Goethals.)

"I had the honor and pleasure of knowing your father, and the opportunity of seeing and understanding some of his splendid traits of character.

"He has been so highly regarded and in so many cases loved by those associated with him, that the human sides of his life shine out as well as do his attainments in his Profession."—(William F. Ford.)

"I believe that every one who had the privilege of knowing your father will feel that he has lost a friend. I have been trying to think of another man in the Engineering Profession who is or has been so universally respected, admired, and even loved as was your distinguished father. My own association with him has been such that I have received a profound impression, not only of his great ability, but of his lovable personal qualities, and I am quite confident that I am simply one of a vast number of engineers and men of other professions who feel the same way."—(Nelson P. Lewis, M. Am. Soc. C. E.)

"I send you my deepest sympathy in your great bereavement, and assure you of the same from every member of our Profession as well as the innumerable personal friends who like myself have felt the high privilege and honor of friendship with your father, a great and noble personality and the most successful and eminent engineer of this generation."—(Frank W. Skinner, M. Am. Soc. C. E.)

"For over twenty years I have watched Mr. Noble's career with interest, and know that, to the younger members of the profession, he has always been a source of inspiration."—(E. G. Haines, M. Am. Soc. C. E.)

"Your father was the acknowledged Dean of our Profession in this country, and I have always felt that it was a privilege and an inspiration to have worked under him."—(T. Kennard Thomson, M. Am. Soc. C. E.)

"I wish to write you a few words of appreciation of your father and the kind fortune that threw me with him at various places, four or five years altogether, in the period 1883-1892. At Snake River and

Second Crossing, I felt that I knew him intimately, and it was a pleasure to renew this friendship some years later at Cairo and in this city, and in the past twenty years I have felt a keen satisfaction in his successes and in the eminence he attained in his Profession, a satisfaction the greater because I knew there was nothing fortuitous about his success, which came as a natural tribute to his high character and great ability, hidden though they were beneath exceeding modesty.

"During the past ten years I have twice been East and on each visit stopped over for the purpose of seeing your father. On both occasions he was out of town and I failed to see him, much to my disappointment. I feared what has happened would happen—that I would not see him again. I wish that the happiness that came to him in his success had not been clouded by your mother's illness. In a letter written some years ago he mentioned his disappointment and sorrow that in later life he could not realize his early ambition of giving her some compensation for the nomadic, and in some ways most unsatisfactory, life that is apt to be the lot of a civil engineer for the first half at least of his active career."—(Sanford Morison.)

"I have known and admired your father for many years, and cannot speak too highly of his ability and of his personal qualities. There are few men in the country to whom the Nation owes a greater debt for large service rendered."—(Charles Whiting Baker.)

"We who have passed the meridian of life have met in our journey many men whom we respect for what they have accomplished, and among them a few whom we respect for what they have done and love for what they are, and your father was one of these latter to me, and I know that he was so regarded by a host of others.

"It was a painful shock to me when I learned, only this morning, that he had joined the silent majority. I feel that I have lost a true friend and the Engineering Profession its foremost American representative."—(Isham Randolph, M. Am. Soc. C. E.)

"As one of the thousands of engineers who knew and loved and admired Alfred Noble, I tender you my sincerest sympathy."—(Onward Bates, Past-President, Am. Soc. C. E.)

APPENDIX.

PAGE.

Resolutions Adopted by the Board of Direction of The American Society of Civil Engineers, June 2d, 1914.

"*Whereas:* By the death of Alfred Noble, the Engineering Profession in America has lost its most prominent member, and

"*Whereas:* Mr. Noble has been connected with this Society for forty years, and has served upon its Board of Direction for nine years, as Director, Vice-President, President, and finally as a Past-President, be it

"*Resolved:* That the Board of Direction of the American Society of Civil Engineers acknowledges the indebtedness of the Profession to this wise counselor, active and tireless worker, who, during his connection with this Board and subsequently, gave ungrudgingly and unselfishly so much of his valuable time for the general good, and be it further

"*Resolved:* That this Board desires to spread upon its records its sense of profound sorrow in the great loss, not only to the Profession of Engineering, but to the world, of one who by his strong and intellectual personality, earnestness of purpose, sterling honesty, and great heart, has set an example for Engineers of the future, and in so doing endeared himself to all with whom he came in contact."

Memoir from *Engineering News*, April 23d, 1914.

"The American engineering profession looked up to Alfred Noble.

"The news of his sudden death will bring a sense of personal loss to thousands of engineers who had never met Mr. Noble personally, but who appreciated the great public service which he had rendered and realized that his great ability and strong personality had done much to raise the engineering profession in public esteem.

"Elsewhere in this issue, in presenting to our readers a recent portrait of Mr. Noble, we have given a brief record of his life and professional achievements. It is fitting, however, that something more should be said in this place concerning the unique public service which Mr. Noble rendered. We believe it is within bounds to say that there are few men to whom the people of the United States owe a greater debt of gratitude for important services rendered at a time of crisis than to Alfred Noble.

"In 1895, Congress was on the point of passing a bill providing for a government guarantee of the bonds of the Nicaragua Canal Co. There was a very strong sentiment in favor of the passage of the bill among members of both the House and Senate. The assurances held out were that the entire cost of building a ship canal across Nicaragua connecting the two oceans would be only $65 000 000. There were, however, in Congress, a few statesmen who were not swept off their feet by the great pressure exerted in favor of the passage of the bill. They put forth the plea that before the United States should lend its credit to the enterprise it should investigate the project through a board of engineers of its own selection.

"This proposition was so entirely reasonable that it sufficed to defeat the bond-guarantee bill. In its stead an act was passed creating the Nicaragua Canal Commission, to be composed of one engineer

from the Army, one from the Navy, and one from civil life. This Commission with a very small appropriation and a very limited time in which to work, was required to report as to the feasibility of the Nicaragua canal enterprise. President Cleveland appointed as the members of that Commission, Colonel (afterwards General) William Ludlow from the Army, Mordecai T. Endicott from the Navy, and Alfred Noble from civil life.

"The situation was one which demanded engineers of ability with sufficient independence to form their own opinions and not be swerved from a straight course by the strong influences brought to bear by the corporation whose plans were under investigation. The report made by this commission showed that the advantages of Nicaragua as a canal route had been greatly over-estimated and that the cost of building a canal there would be far greater than the estimates made by the canal company.

"Those interested in the Nicaragua Canal enterprise adopted every possible means to discredit the report; but they were never afterward able to command a large measure of public support. The sound advice of these engineers saved the nation from lending its credit to a private corporation which, if it had undertaken the Nicaragua work, would have inevitably met failure.

"The second great opportunity of Mr. Noble to render public service came when in 1899 Congress created the Isthmian Canal Commission, with instructions to find the best possible route for a ship canal across the Central American Isthmus. Mr. Noble was appointed one of the members of this commission and he and the late George S. Morison were recognized as its leading engineers. Without doubt, Mr. Noble's large experience, tactful firmness, and ability had large influence in determining the conclusions of the Commission.

"It was this commission which after two years of surveys and investigation recommended that the United States should adopt the Panama route. The experience which has been gained since that time has fully confirmed the wisdom of the recommendations made by that commission.

"It was seven years later when Mr. Noble had the opportunity to render what was, without doubt, the greatest public service of his life. The government had started construction work on the Panama route, and the question came up for decision as to whether a sea-level canal or a lock canal should be undertaken. To advise upon this momentous question, President Roosevelt created an International Commission of engineers, made up of five eminent members of the engineering profession representing foreign countries and eight prominent American engineers.

"As most of our readers will remember, all the foreign engineers and three of the American Engineers united in a majority report advising the construction of a sea-level canal. Five American engineers with Mr. Noble at the head stood out in favor of a lock canal. We say 'Mr. Noble at the head', because from his strong experience in connection with the lock at Sault Ste. Marie, he was better able than any engineer upon the commission to speak authoritatively with respect to the construction and operation of great ship canal locks.

To Alfred Noble's discerning wisdom and independent judgment and to his willingness to stand in a minority in defense of what he believed to be right, the country owes it to-day that it did not undertake what we now know would have been the folly of a sea-level canal at Panama.

"In the struggle which followed the submission of these two conflicting reports, Mr. Noble's ability and strong forceful personality had much to do with the final decision by which those in authority rejected the majority report and adopted that of the minority.

"In reviewing these three great public services rendered by Alfred Noble to the nation it will be universally agreed that his name deserves high prominence in connection with our greatest national engineering work, the Panama Canal. It detracts nothing from the honor due to those who have borne the burden and heat of the days and years during the long period of construction at Panama to give honor to the great engineer whose sound judgment and incorruptible integrity enabled the nation to steer a straight course in undertaking this hugest of engineering feats and avoid the disgrace attendant upon disastrous failure.

"It can be said of Mr. Noble, without fear of contradiction, that he won his way to the foremost position which he occupied as the leading American civil engineer of his time by sheer force of ability. Mr. Noble was always a quiet and modest man, absorbed in his professional work. He never tried to advertise himself nor attempted to put his professional work in any way on a commercial basis. The great responsibilities which were laid upon him came to him because he was a man who inspired confidence in both his ability and his integrity.

"For a dozen years in his early professional career he worked steadily, patiently and practically unknown as an assistant engineer on government work; but it was the experience which he gained there, even more than that which he gained in fields of other engineering work, which in later years, enabled him to solve correctly the vast problems which were laid before him in connection with the Panama Canal enterprise.

"We may not take space to speak at length here of his other great works as an engineer, such as the Pennsylvania terminal in New York City. It is worth while, however, to emphasize the high regard in which he held his profession. Much of his time in recent years had been devoted to work for the benefit of the profession in connection with the engineering societies of which he was a member. Last year, he was President of the American Institute of Consulting Engineers and he had been for two years one of the Managers of the American Society of Mechanical Engineers. In 1903 he was President of the American Society of Civil Engineers.

"One of the last pieces of work which he undertook and to which he had devoted a great amount of time and energy was in connection with a joint committee of the national engineering societies organized to frame a model code for the registration of engineers. Mr. Noble was the chairman of this committee and he devoted to the task a great amount of painstaking thought. It was characteristic of the man that

with all the great responsibilities laid upon him, he was willing to give liberally of his time and energy to benefit the members of his profession."

MEMOIR FROM *Engineering Record,* APRIL 25TH, 1914.

"Alfred Noble, master civil engineer, universally respected because of his ability, loved and honored by those who knew him, died in New York, April 19. Apparently in vigorous health, despite the attainment of the Scriptural years, he was stricken rather suddenly during the first week in April. On April 9 it was deemed advisable to perform an operation and though he rallied for a time his condition hardly became encouraging.

"His early career and his experience until the close of his work on the Pennsylvania Railroad's improvements in New York were recounted admirably by Dr. Rossiter W. Raymond, secretary-emeritus of the American Institute of Mining Engineers, when the John Fritz medal was presented to Mr. Noble on Nov. 10, 1910.

"Since the close of the period covered by that biography, Mr. Noble has been in general consulting practice, serving also on retainer as consulting engineer for the New York Board of Water Supply. For this service his experience in tunneling, in the building of masonry structures and the examination of foundations proved invaluable. His knowledge of foundations, too, brought him a most important retainer last year from the Federal Government. Great difficulty was experienced in the construction of a drydock at the Naval Station, Pearl Harbor, Hawaii, ending in the eruption of the bottom and the stoppage of the work. Mr. Noble was sent to Hawaii to advise as to the method by which the work should be completed.

"Among the other large works which had recently engaged his attention was the enlargement of the Welland Canal, upon which he reported for the Canadian Government, thus linking his name with another of the world's great waterways. The Public Service Commission of the First District, New York, which is building a $165 000 000 subway system, also called him into consultation.

"Many honors conferred upon him by his colleagues are recounted in Dr. Raymond's biography. Following his selection for the John Fritz medal, the highest American honor for an engineer, he was selected an honorary member of the Institution of Engineers of Great Britain, a distinction enjoyed by no other American engineer. In 1912 the Franklin Institute awarded him the Elliott Cresson medal.

"In recent years he was particularly interested in anything affecting the status of engineers and it was largely through his influence that the American Institute of Consulting Engineers entered so actively into public affairs. This organization within the past year addressed communications to the President of the United States, the Governor of New York, and the Mayor of New York City, urging the appointment of engineers to such public offices as their training particularly fitted them for. Activity was displayed, too, on the licensing of engineers and Mr. Noble, as a representative of the Institute, journeyed more than once to Albany to plead for proper legislation."

Biography of Alfred Noble by Dr. Rossiter W. Raymond, on the Occasion of the Presentation of the John Fritz Medal to Mr. Noble at the House of the American Society of Civil Engineers on November 30th, 1910.

"Alfred Noble was born in 1844 at Livonia, Wayne County, Michigan. His father, Charles Noble, was a farmer, like most of our pioneer settlers in the West. But he and his fellow farmers made of their adopted State a notable center of intelligence and industry. When Alfred Noble was born Michigan had been but seven years a State of the Union; yet she had already begun that course of material and intellectual culture which soon placed her in these respects abreast of the most favored sections of the country. A splendid public-school system, crowned with numerous excellent high schools and colleges, and a great University represent one of these achievements. What wonder that this farmer's boy, taught in the district free school and the village high school, dreamed of the University, as the gateway to an honorable professional career!

"Yet in him, as in so many American boys at that time, and, I doubt not, at *this* time, there was one passion stronger than personal ambition—the love of country. When the first call to arms was sounded by Lincoln, Alfred Noble was too young to be accepted. But in August, 1862, when he was eighteen years old, answering the more important call, and volunteering, not for a brief, easy and victorious campaign, but for long, hard service of the Union, he enlisted as a private in the Twenty-fourth Michigan regiment.

"Oh, those Michigan farmers' sons, who far from home, and defending not the soil of their own State, but the wider, grander cause of the nation, grimly, loyally, saw the thing through, until the last straight furrow had been plowed and the last field sowed for the harvest of peace! In the West and in the East are the honored graves of a host of them who fell by the way; but not who survived. The Twenty-fourth Michigan belonged to the sorely tested and grandly faithful Army of the Potomac; and through three terrible years Alfred Noble served with his regiment in the famous First Corps of that Army, consolidated, after Reynolds fell at Gettysburg, with the Fifth Corps under Warren. During the first day at Gettysburg, this regiment lost 300 in killed and wounded, out of the 460 who went into the fight. The figures speak for themselves, and what they mean, perhaps only an old soldier fully understands.

"At the close of the war, Mr. Noble was discharged with the rank of sergeant, and his patriotic duty having been well done, he resumed the purpose of his youth. His record as a veteran, together with his proved character and capacity, secured for him a clerical position in the War Department, which he filled for more than a year, earning the money, and by diligent use of his leisure time, acquiring the necessary scholastic preparation for a university course. In this way he more than fulfilled the requirements for a freshman, and in September, 1867, he was admitted as a sophomore into the University of Michigan, where he was graduated in June, 1870, as civil engineer. Yet, during these three years, he was obliged to earn his expenses as a student by much outside work, as recorder on the U. S. Lake Survey,

as clerk, and afterward as assistant engineer in river and harbor work on the east shore of Lake Michigan.

"After his graduation, Mr. Noble continued his work on the harbor surveys conducted on Lakes Michigan and Huron by the U. S. Corps of Engineers, and in 1870 was placed in local charge of the improvement at Sault Ste. Marie. This position he retained for twelve years, a period covering the construction of the great masonry lock at the Sault, at that time by far the largest canal lock in the world. In 1882, after its completion, Mr. Noble resigned his position to become resident engineer for Mr. G. Bouscaren in the construction of the truss bridge over the Red River at Shreveport, La. Early in 1884, he was appointed general assistant engineer of the Northern Pacific Railroad, of which the late General Adna Anderson was then chief engineer. During the next three years Mr. Noble had charge of the building of important bridges, including the truss bridge, with draw, over the Snake River, near its junction with the Columbia, the bridge over Clark's Fork of the Columbia River, the bridge over St. Louis Bay on Lake Superior, and also the foundation and construction of the Marent Gulch viaduct, near Missoula, Montana.

"In August, 1886, Mr. Noble removed to New York, to become and remain until July, 1887, resident engineer in the erection of the Washington steel arch bridge over the Harlem River, under the late W. R. Hutton as chief engineer. He then took charge for Messrs. Morison and Corthell of the erection of the bridge at Cario, on the Ohio River. This brought him into association with the late George S. Morison, whom he served in the erection of the great cantilever bridge over the Mississippi at Memphis, and other bridges at Bellefontaine, Leavenworth, and Alton. Mr. Morison's high opinion of his colleague and assistant is matter of record.

"It is worthy of note that in this continuous activity of nearly 25 years, Mr. Noble had neither won nor sought a newspaper reputation. He had never been advertised as chief engineer of anything. He had merely done his part, loyally and efficiently, in every enterprise with which he had been connected, impressing himself upon his superiors and associates as a man of thorough training, wide experience and absolutely trustworthy character. A reputation thus acquired wears well. Engineers were not surprised when, in April, 1895, he was appointed by President Cleveland a member of the first Nicaraguan Canal Commission. Of Mr. Noble's work in that capacity, and of its important results, I shall not here speak. Nor can I, in the brief time at my disposal, discuss his work as a member of the Isthmian Canal Commission of 1899, which resulted in the adoption by our Government of the present scheme of the Panama Canal. When this subject came up for discussion in Congress, Major W. H. Wiley, a member of the House of Representatives, presented a letter from Mr. Noble, stating clearly and tersely the argument in favor of a lock-canal. This letter was printed in the 'Congressional Record', and is said to have influenced decisively the action of both Houses.

"But I must go back a little in order to mention what seems to me to be one of the greatest, if not the very greatest, of the engineering investigations with which Mr. Noble has been connected. I refer to the labors of the U. S. Deep Waterway Commission appointed in

1897 to conduct surveys for a deep waterway from the Great Lakes to tide-water. This body spent half a million dollars in its investigations; fixed 21 feet as the most economic depth; proved the most practicable route to be *via* Lake Ontario and the Oswego and Mohawk rivers; examined by borings, etc., every part of that route, and determined the nature and cost of the work (in every particular except the price of the private property to be purchased or condemned for it) so accurately that a contractor might safely have based his bid for any section upon its report. I have never encountered in the literature of engineering, and I doubt whether that literature contains a discussion so thorough, exhaustive and conclusive. Before that report had been prepared the estimates of engineers—I mean such guesses as engineers sometimes permit themselves to make—had varied by a hundred million dollars as to the cost of the proposed waterway; and it is my impression that even this wide variation did not bring them within a hundred millions of the truth. Be that as it may, the report of this commission, published in 1900, will always remain a monument of professional thoroughness and a model for professional imitation.

"Among other engineering enterprises with which Mr. Noble was connected at this period, I may name the great seawall, built to protect the City of Galveston, Texas, against a recurrence of the disastrous flood of 1900, and the bridge across the Mississippi at Thebes, Ill., which was erected by him in partnership with Ralph Modjeski. Moreover, he has been employed as consulting engineer in connection with the difficult problems presented by the foundations of some of the lofty office buildings of New York City—structures which certainly need to be planned with more care and knowledge than ordinary architects and builders bring to such tasks.

"But the latest of Mr. Noble's labors is also, perhaps, the most important. He was appointed in 1902 a member of the Board of Engineers directing the operations of the Pennsylvania Railroad Company (through auxiliary corporations in New York and New Jersey) in tunneling under the North and East rivers, and under the borough of Manhattan, establishing a great railway-station on Seventh Avenue. The plans approved by this board, and executed under its direction, have been so fully described in recent papers before the American Society of Civil Engineers and the American Institute of Mining Engineers as to need no recapitulation here. Mr. Noble, besides serving as a member of the board, was, as chief engineer of the East River Division of the Pennsylvania, New York and Long Island Railroad, directly in charge of the construction of the tunnels from Seventh Avenue under Manhattan and East River to the portals on Long Island, the approaches from the east, and the immense terminal yard at Long Island City. This part of the great undertaking is reported to have cost more than $30 000 000. One thing I believe I may safely say—that the difficulties encountered in the quicksands and the decayed and fractured gneiss pierced by the tunnel under the East River were much more serious, though much less widely reported in the newspapers than those presented by the glacial silt which forms the bottom of the Hudson. True to his

record, Alfred Noble advertised neither his trials nor his triumphs, but simply finished his work without interlocutory appeals to the public. At the end of 1909, that work being done, he resigned his position as chief engineer. The directing board of engineers, having concluded its work, had closed its offices, and, I believe, ceased to exist six months earlier. Such a quiet, business-like, unboastful termination of a colossal engineering enterprise was worthy, in its simplicity, of the great men who planned it and the great men who carried it out.

"Perhaps I may be allowed to say that, in this particular work, Mr. Noble came nearest to the heart of us mining engineers. For several years he and his associates made of New York and its vicinity one of the greatest mining camps in the world. True, in all their tunneling they were only making a hole—not extracting gold or silver or copper from it. Yet, can we say more for most of our mining tunnels? Do they not too often leave us with the hole only as a net result? After all, we mining engineers do not control the commercial results of our borings and excavations. Yet we are often unjustly held responsible for such results, and we cannot but congratulate this mining engineer, whose employers ask only that he shall put his job through and will look for their dividends afterwards, not to the contents of the hole, but to the use of the hole itself. In other words, Mr. Noble has been, in this work, an ideal mining engineer, unhindered by the assayer, the millman, the economic geologist, the mining law or the stock market. We greet him, not without a touch of envy, as our brother!

"In this connection let me voice the opinion of mining engineers as to the manner in which Mr. Noble conducted, under land and sea with the minimum of disturbance to the surface, his extensive operations. Some of us (I among the number) have suggested from time to time, with the freedom of those who were not responsible for the results, ways in which this work could be still more quietly and safely done. But all of us agree that in these respects such work never has been better done and we have sense enough to admire and praise the man who directed it.

"Mr. Noble's merits have been recognized in various ways by those whose judgment he would most highly value. In 1895 his university conferred upon him the honorary degree of Doctor of Laws, an honor which was repeated in 1904 by the University of Wisconsin; in 1898, he became president of the Western Society of Engineers; in 1903, he was elected president of the American Society of Civil Engineers (of which he had been made a junior in 1874 and a member in 1878). His membership in the ancient Institution of Civil Engineers of Great Britain certifies his professional standing abroad. And we have elected him a member of the Engineers' Club of New York City in testimony that he is not only an eminent engineer, but a congenial companion and a true friend. Yet I fancy that not one of these distinctions—perhaps not all of them put together—will outweigh in his esteem the honor conferred upon him to-night, with the hearty professional approval, and the personal esteem and affection of American engineers!"

LETTER FROM WILLIAM H. TAFT, EX-PRESIDENT OF THE UNITED STATES.*

"I am very sorry to hear of the death of Alfred Noble. He was the dean of American engineers and has left a record of brilliant usefulness upon which it is inspiring to dwell. I had at one period much official relationship with him and came to respect him most highly as a man and as an engineer. His professional advice in respect to the type of the Panama Canal and the security of the foundations of the Gatun Dam was followed by the Government and has been vindicated completely by the event."

LETTER FROM GEORGE W. GOETHALS, M. AM. SOC. C. E., COL., CORPS OF ENGRS., U. S. A., CHAIRMAN AND CHIEF ENGINEER, ISTHMIAN CANAL COMMISSION.*

"I have for many years held Mr. Noble in high esteem both as a man and as an engineer. The country is under great obligations to him for his wise and far-sighted course in relation to the Panama Canal. As a member of the International Board of Consulting Engineers, assembled by President Roosevelt in 1905, he threw the weight of his long experience and acknowledged engineering ability in favor of a lock as against a sea-level canal and wrote the report of the minority members of that body, in which the plan of the canal as constructed was outlined. As a member of a special commission of three sent by President Roosevelt to the Isthmus in 1907 to make a special investigation of the lock and dam sites, his signature to a report declaring the foundations safe and stable had great effect in reassuring the public confidence."

LETTER FROM SAMUEL REA, M. AM. SOC. C. E., PRESIDENT, PENNSYLVANIA RAILROAD.*

"It was with the deepest regret that I learned of Mr. Noble's death. My relations with him had been very close for many years during our association on the important New York tunnel work for the Pennsylvania Railroad. He was a man for whom everyone entertained the highest respect—not only for his personal ability and talent but for his modest and lovable personal characteristics. In my judgment the profession has sustained a great loss in Mr. Noble's death."

LETTER FROM E. L. CORTHELL, M. AM. SOC. C. E.*

"From twenty-five years of professional association with Alfred Noble my judgment is that, considering all his sterling qualities, he has had very few equals in those solid, reliable traits of character that make for usefulness of a high order to the civil engineering profession and to the world at large."

LETTER FROM J. WALDO SMITH, M. AM. SOC. C. E., CHIEF ENGINEER, BOARD OF WATER SUPPLY, NEW YORK CITY.*

"In the entire engineering profession there is probably not another man whose death would be more sincerely mourned than that of Alfred Noble. Above everything, he was a man and beloved by all who came in contact with him, a man to whom every one in trouble might go

* To *The Engineering Record.*

and gain something from the wealth of his experience. He was the most conscientious engineer I have ever known. He never rendered snap judgment, even on matters of small importance. Any advice given or judgment rendered was always the result of the most careful consideration. Material things of the world were, with him, always a minor consideration and he repeatedly refused lucrative engagements for the sole reason that he felt that he could not give them the study and the attention which they demanded. His advice was sought not only by the young, struggling engineer, but also by those of wide experience, not in engineering matters alone, but in any matter where experience and good judgment were desirable. He had the greatest breadth of mind and his keenness of vision caused him to see problems in their true light. His acquaintance was world-wide, and his death will be regretted by thousands of engineers who have at some time or other come under his influence. My intimate association with the late Charles L. Harrison brought me into close touch with Mr. Noble and I cannot express by words my appreciation of his influence and of his help. I trust that his influence may live after him."

LETTER FROM JOHN F. WALLACE, PAST-PRESIDENT, AM. SOC. C. E.*

"In the passing away of Alfred Noble our Profession has lost one of its best and highest representatives—the leader in his special work, true to his friends, a gentleman, a man in all that the word implies. He has left a vacancy in our ranks that cannot be filled."

PROPOSED MEMORIAL TO ALFRED NOBLE;
TRIBUTES FROM BROTHER ENGINEERS AND CO-WORKERS.

At the meeting of the Board of Direction of the American Society of Civil Engineers on June 2d, 1914, the Secretary presented the following report:

"TO THE BOARD OF DIRECTION
OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS:

"The undersigned, appointed at the last meeting to consider the matter of a proposed Memorial to the late Alfred Noble, Past-President, Am. Soc. C. E., beg leave to report as follows:

"That, in the opinion of your Committee, such a Memorial is desirable, and should be undertaken by this Society, and should be on no small scale. Your Committee suggests that the Civil Engineers of the world be asked to subscribe to the funds for this purpose, and that the Memorial take the form of an appropriate bronze statue to be erected in some suitable place.

"In view of the fact that much of Alfred Noble's professional practice was National in character; that he was an adviser of Presidents, and consulted in some of the most important engineering work of the country, your Committee believes that such a recognition of him personally, and of the Profession of Engineering, would meet with universal approval.

"It is therefore the opinion of your Committee that the Capital of the Nation is the proper location for such a statue, and that the

* To *The Engineering Record.*

Congress, or other body in authority, be requested to provide a suitable site. Your Committee, therefore, makes the following recommendations:

"(1) That this Society undertake the erection, in a suitable location, of a Statue to the memory of Alfred Noble—the Engineer and the Man.

"(2) That the Board of Direction immediately set aside the sum of one thousand dollars ($1 000) as the first subscription toward the necessary funds.

"(3) That the funds for this purpose be secured by subscription from the Engineers of the World.

"(4) That a Committee of five be appointed by the Board, in whose hands the carrying out of the project be placed.

"Yours respectfully,

"T. KENNARD THOMSON, *Chairman,*
J. H. EDWARDS,
CHAS. WARREN HUNT."

The recommendations of the report were adopted as the action of the Board, and the President was authorized to appoint a Committee, with power to carry out the suggestions made.

The President subsequently appointed as such Committee: Onward Bates, Chairman, Robert Moore, Samuel Rea, Samuel H. Hedges, F. H. Newell, and Chas. Warren Hunt, Secretary.

RESOLUTIONS ADOPTED BY THE JOINT CONFERENCE ON UNIFORM METHODS OF TESTS AND STANDARD SPECIFICATIONS FOR CEMENT.*

"The Joint Conference on Uniform Methods of Tests and Standard Specifications for Cement hereby records with profound sorrow the irreparable loss sustained in the death of its Chairman, Mr. Alfred Noble, and its deep gratitude for the privilege of having known and been associated with one who was the exponent of the highest ideals as a man, as a citizen and as an engineer.

"Mr. Noble has given much to the engineering profession and his unselfish work in the development of the methods of testing cement has been of incalculable value. Many of his contributions to the Engineering Profession, to be found in the *Transactions* of the American Society of Civil Engineers, relate to this branch of engineering. He was a member of the Special Committees of this Society which presented reports on Uniform Methods of Tests of Cement in 1885 and 1912.

"The members of this Conference find themselves unable to adequately express their loss. This intimate association with Mr. Noble has left in the memory of each member an indelible impression of his patient, sweet, endearing nature, of his simple, forceful, dignified personality, and of the grandeur of his character."

* The Joint Conference is composed of Messrs. Arthur P. Davis, Olaf Hoff, Richard L. Humphrey, Asa E. Phillips, Clifford Richardson, George F. Swain, George S. Webster, and Rudolph J. Wig.

MEMOIR FROM THE JOURNAL OF THE AMERICAN SOCIETY OF MECHANICAL ENGINEERS, MAY, 1914.

"There passed away on April 19, 1914, in New York City, Alfred Noble, an esteemed member of our council and a man whose loss will be deeply felt and deplored not only by the engineering profession of which he was one of the most distinguished members, but by everyone who had the good fortune to know him.

"He had a very interesting career, and the story of his life, if adequately written, would be typical of that of many of the great men and builders of this nation.

"He was born August 7, 1844, at Livonia, Wayne County, Michigan, where his parents resided on a farm. His early education was received in the district school of his native place, and during his spare time he worked on the farm.

"In 1862, when only 18 years of age, he enlisted in the Civil War in the 24th Michigan Volunteer Infantry. From that time until 1865 he served in the Army of the Potomac, taking part in all of the hard and desperately fought battles which that army engaged in against Lee and Stonewall Jackson. At Gettysburg his regiment lost a very large percentage of its numbers. At Chancellorsville, it was by the merest accident that his brigade was not captured by Stonewall Jackson's men, but he was lucky in serving through the war without being wounded, and was mustered out of the service in June 1865 with the rank of sergeant. He then prepared to enter the University of Michigan, and in 1867 became a sophomore, graduating in 1870, with the degree of C. E. He received the degree of LL.D. from his alma mater in 1895, also from the University of Wisconsin in 1904.

"From 1868 to 1870 he was assistant engineer on river and harbor work on the Great Lakes. From 1870 to 1872 he was in charge of improvements on St. Mary's Falls Canal and St. Mary's River. During this time the first great masonry lock at the Sault, then by far the largest canal lock in the world, was built. On completion of this work he became resident engineer on the construction of an important bridge at Shreveport, La., over the Red River.

"From 1883 to 1886 he was general assistant engineer on the Northern Pacific Railroad, and from 1886 to 1887 resident engineer on the construction of the Washington Bridge over the Harlem River, at that time the largest arch bridge in existence.

"From 1887 to 1894 he was resident engineer on the construction of several very large and important bridges over the Mississippi at Memphis and Alton, over the Missouri at Bellefontaine and Leavenworth, over the Ohio at Cairo.

"He was appointed a member of the Nicaragua Canal Board by President Cleveland in 1895. This board visited Central America and examined the route of the Nicaragua Canal and also the Panama Canal and then returned to the United States, completing its work November 1, 1895.

"In June 1899 he was appointed by President McKinley a member of the Isthmian Canal Commission which was charged with the selection of the best canal route across the American isthmus, and it

has been substantially on the route selected by this commission that the Panama Canal has been constructed. While on this commission, Mr. Noble with his colleagues visited Europe to examine the existing canals there, and the data which the French Canal Company had in Paris, and also made several trips to Central America to examine more fully the various canal routes.

"In 1905 he was appointed by President Roosevelt a member of the International Board of Engineers to recommend whether the Panama Canal should be constructed as a sea-level or a lock canal. This board consisted of thirteen members, of whom five were nominated by foreign governors. Mr. Noble was one of the minority of five Americans who recommended the adoption of the lock plan. Their views were adopted by the Government and the canal has been built in accordance with their recommendations. In March 1907 he was one of the three appointed by President Roosevelt to visit the Panama Canal to investigate the conditions regarding the foundations of some of the principal structures. This duty was completed in a few weeks. He was obliged to decline a similar appointment two years later.

"From the very inception of the plan by this country to build an Isthmian Canal, and from the commencement of the preliminary investigations and surveys, to the adoption of the final plan and the beginning of the actual construction of the Panama Canal, Mr. Noble was continuously identified with the project and deserves as much credit for the solution of the engineering problems as any other one who has been connected with this great work.

"In July, 1897, he was appointed by President McKinley, a member of the United States Board of Engineers on Deep Waterways, which made surveys and estimates of cost for a ship canal from the Great Lakes to deep water in the Hudson River.

"In November, 1901, the city authorities of Galveston, Texas, appointed Alfred Noble along with Henry C. Ripley and General Robert, as a board of engineers to devise a plan for protecting the city and suburbs from future inundation. They recommended the building of a solid concrete wall over three miles long and seventeen feet in height above mean low water, the raising of the city grade, and the making of an embankment adjacent to the wall; the whole to cost about three and a half million dollars, which plan has since been carried into effect.

"From 1902 to 1909 Mr. Noble was chief engineer of the East River Division of the New York extension of the Pennsylvania Railroad, and was in entire charge of this most difficult piece of work, involving as it did, a very accurate survey across Manhattan, and the construction of the foundations of the Pennsylvania Station, of the land tunnels, and of the East River tunnels which were very troublesome.

"Since 1909 he has been engaged in general practice as a consulting engineer, the firm name being Noble and Woodard. Probably the most important work dealt with was in relation to the dry docks built for the United States Government near Honolulu. He was also for a time consulting engineer to the Quebec Bridge Board, also consulting engineer for the Board of Water Supply, New York City,

and for the Public Service Commission of the First District of the State of New York.

"He was a past-president of the Western Society of Engineers, the American Society of Civil Engineers, and the American Institute of Consulting Engineers. He was elected to the Council of this Society in 1912 and had served several years on the Library Committee.

"In 1910 he was awarded the John Fritz Medal for notable achievements as a civil engineer, and in the same year was elected an honorary member of the Institution of Civil Engineers of Great Britain, a distinction which no other American has had. In 1912 he received the Elliott Cresson Medal of the Franklin Institute in recognition of his distinguished achievements in the field of civil engineering.

"Mr. Noble was deeply interested in anything affecting the status of the engineering profession. His unfailing good humor, his kindliness and sweetness of disposition, his sound common sense and good judgment, his youthful mentality, his quick and very sure perception, and his modesty, invariably impressed his colleagues with whom he worked on many committees, and commissions in which he was so active.

"He possessed a combination of strength, gentleness, tact and discernment rarely met with. He was universally respected by all who had any business dealings with him. The plain workman, the man with the pick and shovel, the contractor under him, the highly trained technical engineer, or the president of a great corporation, all appreciated the nobility, simplicity, and rugged honesty of his character. His personality was such as to evoke the faithful and enthusiastic loyalty of his subordinates, and the deep, strong, and lasting affection of all those who were honored with his friendship.

"At the funeral services on the evening of April 21, the Society was represented by Jesse M. Smith, Past-President and member of the Council; Leonard Waldo, Chairman, and E. G. Spilsbury of the Library Committee, Charles Whiting Baker, Rudolph Hering, J. Waldo Smith, C. M. Wales, W. L. Saunders, and Calvin W. Rice, Secretary."

Resolutions of the Council of the American Society of Mechanical Engineers, Adopted October 9th, 1914.

"In the death of Mr. Alfred Noble, the engineering profession has lost one of its greatest members, one of its wisest associates, and one of its most modest scientists.

"Mr. Noble was a man of generous impulses, always interested in the success of younger engineers, always ready to help them with advice, and to put before them an opportunity for their success. He was without the slightest professional jealousy, and so in love with his chosen calling that he always hailed the achievements of others with delight because engineering had by them been advanced and the world benefitted. His personality was most charming and The American Society of Mechanical Engineers will long miss his delightful talks and wise advice at its Council meetings, where he was a most welcome member. He may be aptly described as a lovely man, full of

gentleness and dignity, and yet possessing a forceful character which fitted him so well as a cherished adviser.

"It may not be generally known that Mr. Noble had an influence in the decision of Congress to abandon the sea-level plan and adopt the lock system for the Panama Canal. The subsequent events have shown the wisdom of Mr. Noble's advice. A member of Congress and a personal friend of Mr. Noble asked him to state his reasons for advising the lock system in the form of a letter. This was done in a most concise form and was read in the House of Representatives, and thus became incorporated in the Congressional Record, with the result that it convinced the members, and by a large majority they adopted the lock system. Copies of the Record marked at Mr. Noble's letter were given to each Senator, and the argument was equally convincing, so that the Senate confirmed the House action by a large majority.

"A glance at Mr. Noble's history will be most edifying to a young engineer as it will be gratifying to his hosts of friends.*

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"He was married May 31, 1871, to Miss Georgia Speechly, of Ann Arbor, Michigan. They had one son, Frederic Charles, a graduate in Engineering of University of Michigan, 1894, now following his profession in New York City.

"There is little to add to this epitome, but it shows the forceful character of Mr. Noble throughout. He won the various honored and honorable positions he so ably filled by merit and perseverance, and his career, cut short in this untimely manner, is an encouragement to every young engineer and a stimulus to the exercise and cultivation of those manly and fearless qualities in the possession of which Mr. Noble so excelled and which have so firmly established him in the affections and admiration of all engineers."

Resolutions Adopted by the United Engineering Society Library Board, on May 7th, 1914.

Alfred Noble.
1844-1914.

"Earnest boy—Youthful patriot—Full-serviced soldier for the Union—Determined student—Teacher—Discerning and courageous engineer—Safe adviser in great enterprises—Receiver but not seeker of highest honors—Friend and up-builder of young men—Guardian of the honor of the Engineering Profession—Organizer of this Library Board.

"Others have been impressed with the compass of Alfred Noble's scientific imagination, whether shown in the mid-Pacific docks, or in the choice of a continental passageway or in the water system of a great metropolis, or the sinuous frames on which the commerce of our nation is woven, will spread afar the record of his great engineering career; but it is for us, his messmates at the club, his comrades in our council for the diffusion of most useful knowledge, to see as well as we may at this close vision and with dimmed eyes, and to testify to that great soul which we know was with us, and which we shall increasingly feel has gone.

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* Here there is a brief history of Mr. Noble's career, which will be found on pp. 1380-82.

"He earnestly believed the spreading branches of this tree which he planted here with us would bear increasingly richer food for the minds of men, and that as the centuries pass on future generations will say that together we builded better than we knew. To quote his own words—

"'Our Library is coming out all right. I will help you all I can. I shall not be with you at the next meeting for I have some important work to do. I go a-fishing'

and with that smile on his face, which has since become a benediction, we saw him for the last time.

"Would that some sculptor had preserved for us, and for those who knew him less, the unfathomable smile, the gentle humor, the roused dignity, the life below the outward surface of that fine face.

"His three score and ten years of life of greatest attainment seemed the embodiment

"'Of toil unsevered from tranquillity;
Of labor that in lasting fruit outgrows
Far noisier schemes, accomplished in repose
Too great for haste, too high for rivalry.'

"To his tenderly loved wife and to their son, to whom the Angel of Light has brought the message of final promotion, we speak our deep sympathy in this grievous sorrow of separation.

"We are comforted to think that in their spiritual hearing, as in ours, must be echoing the words

"'Well done, good and faithful servant, enter thou.'

"'There lies not any troublous thing,
Nor sight nor sound to war against thee more,
For whom all winds are quiet as the sun,
All waters as the shore.'"

FROM E. W. STERN, M. AM. SOC. C. E.

"Replying to your letter of May 1st, I willingly send you whatever information I have regarding Alfred Noble.

"I gave most of the facts I knew to *Engineering Record* and this information was embodied in the issue of April 25, 1914, copy of which I enclose you herewith.

"Dr. Raymond of the American Institute of Mining Engineers is the one man who knows Mr. Noble's war history, and I believe he could give you more facts than what are given in the above article.

"As you know, Mr. Noble was very loath to speak of his war experiences, and it was during the course of the last three years of rather close acquaintanceship with him that from time to time I was able to glean a few incidents. It appeared to me that he wanted to forget the Civil War. The humorous side he was more inclined to dwell upon. He never wore a Grand Army button, to my knowledge. He told me the following incidents:

"Some years ago, when he was Resident Engineer on the Memphis Bridge, a man wearing a Grand Army button, claiming to have been a Colonel or Brigadier General during the War, came to see Mr. Noble about some matter or other in connection with some material

he was selling—I think it was paint—'He told me', said Mr. Noble, 'That he had been at Chancellorsville', and mentioned a certain incident which I knew did not agree with the facts, which I told him. He seemed rather astonished at my information on this point, and asked me how I knew, and I told him that I was there. He asked me to what Army Corps I belonged, and what rank I held. 'I am the last surviving member of it', I said. He seemed curious to know, and I replied, 'The great Corps of Privates'.

"On another occasion, in talking with him about the requisite qualities of a good soldier, he said, 'Ability to withstand hunger, fatigue, and hard marching, were very essential qualities, but to be a good runner was also often a very useful attribute.' At Chancellorsville, had his brigade not been swift-footed, including his Brigade Commander, they would have been encircled in Stonewall Jackson's flank attack, and along with 10 000 others been made prisoners of war.

"I was often impressed with a quality he had of being able to listen very carefully and then to give an opinion very quickly afterward. This innate faculty, together with his splendid ability in handling men was no doubt developed by his war experiences. He frequently made use of a remark which impressed me on this point. In speaking of individuals he would sometimes say that such a one 'had the qualities of mind essential in a capable military leader; namely, of being able to listen carefully, think quickly, act promptly, and be nearly always about right.'

"Of Stonewall Jackson he said that there was no Corps Commander in the Northern or Southern Armies who was to be compared with him. He stood in a class by himself.

"Of the war, he said that it might have been avoided.

"Of course you know more of him personally than I do, so I shall not go any further."

From Edgar O. Durfee, Esq.

"Yours of the 25th inst. asking me to give you certain data relative to the late Alfred Noble was received.

"I have known Alfred Noble as long as I have known anybody. His farm home was about a mile from the farm on which I was brought up, and I saw him very often. Some of the time we attended the same district school and the winter of 1861 attended the graded school in Plymouth where we were in the same class in higher algebra. From his earliest school days he always excelled in all of his studies. He was very studious, and as a boy was the same as a man, always truthful and always lived up not only to the letter but to the spirit of his promises. One little thing illustrates his character in that regard. He was eighteen years of age on the seventh day of August, 1862, on which day he enlisted in Company C, Twenty-fourth Michigan Infantry. His mother was thoroughly imbued with the idea that card playing was the greatest vice in the army—although we of the rank and file learned that the games played there were only euchre and old sledge. She asked him not to play cards, and he promised her he

would not play cards while in the army. He lived up to this promise strictly, although he watched the boys play and learned more of cards than most of them, and as much as any of them. As soon as he was discharged from the army, July 1st, 1865, he joined with the other boys in the card games above referred to.

"His life on the farm was the usual boy's life, hard work and very little play. His school life until he went into the army was confined to the district school in his neighborhood and the graded school in Plymouth some three and a half miles away. In the army he did his duty at all times and during a part of 1864 was an orderly on the staff of General Warren then commanding the Fifth Army Corps and was discharged as a sergeant. He returned home and went to work on his father's farm in the harvest field. This work, he informed me afterwards, did not strike him as his particular line of work and he was anxious to do better. He went, I think, in July to Washington, D. C., and obtained a clerkship in the War Department where he remained two years, in the meantime studying so that in 1867 he entered the University of Michigan in the Engineering Department—Sophomore year—graduating in 1870. During his first two years as I learned, although I was not in college with him, he did not attend the University more than half of the time, being employed at other times in river and harbor work carried on by the Government in order to procure means to pay his expenses in college. He spent his whole time in college during his senior year. Probably all of the professors in his department at that time are gone and I do not know of any of his classmates now living who are handy to get at. My impression is that Justice Day of the United States Supreme Court was a classmate and intimate friend of Mr. Noble's and he could give you more valuable information as to his college course than I can.

"I think he was the best boy and man I ever knew, taking him all in all. He was very quiet, not given to boasting, was a warm friend, and had as fine a sense of humor as any person I ever knew. I am sure that everybody who came in contact with him as a boy and man was his friend.

"I think the foregoing will give you sufficient points in his early life as well as any particular details that you will want. You can no doubt work it out so that it will be readable."

Letter from Joseph Ripley, M. Am. Soc. C. E.

"Replying to your favor of May 25, 1914, I will send you a synopsis of Mr. Noble's life at Sault Ste. Marie, Mich., at as early a date as possible.

"As you are undoubtedly aware, Mr. Noble served in the Civil War, enlisting on his eighteenth birthday as a Private in one of the Michigan regiments. After the close of the War, he served as a clerk in Secretary Stanton's office for a couple of years. I suggest that you write to Mr. Noble's cousin, Mr. W. Durfee, who enlisted at the same time he did and who has been Judge of Probate in Detroit, Mich., for over thirty years, as he can give you full particulars of Mr. Noble's army service."

Sketch by Joseph Ripley.

"Alfred Noble, the pre-eminent engineer and man was born at Livonia, Michigan, August 7, 1844. His parents, Charles and Lovina D. Noble, were prominent among the intelligent farmers who settled in that part of the State. Mr. Noble's fine physique was well developed by his boyhood life on a farm and his educational training was well grounded during the short term attendance of the country district and village schools. Enlisting as a private in the 24th Michigan on his eighteenth birthday, Mr. Noble served three years in the Army of the Potomac, taking active part in many battles. At Gaines' Mills he was in the rear guard protecting the retreat of the federals and five times during the day, while busily firing, found himself with a squad of about six men at the very apex of the defense with the rebel advance charging within fifty feet of them. One night while on sentry duty after a hard day's march, and with his system filled with malaria, combined with the sleepiness of a growing youth, drowsiness overcame him, but an alert comrade on the next post awakened him just a moment before discovery by the officer on round of duty and thus escaped being shot the next morning with another sentinel who had been found sleeping at his post in the presence of the enemy. Mr. Noble was mustered out as sergeant. His army service also included nearly two years' service as clerk in the office of Secretary of War Stanton, and while in Washington he prepared for entrance in the University of Michigan. He attended class recitations fourteen months of the four years' course, as he was employed during the working season of each year on Government work, principally at Milwaukee, but also at several harbors along the eastern shore of Lake Michigan and on the survey of Lake Superior, at a salary during his junior and senior years of one hundred and fifty dollars a month. Mr. Noble received the degree of Civil Engineer with the class of 1870. The honorary degree of LL.D. was conferred by the University of Michigan in 1895 and by the University of Wisconsin in 1904. He was placed in local charge of the proposed canal and river improvements at Sault Ste. Marie, Michigan, in the fall of 1870. Mr. Noble married Miss Georgia Speechly of Ann Arbor, Michigan, on May 7, 1871, and their only child, Mr. F. C. Noble, is a distinguished engineer now in charge of one of the five field divisions of the subway construction by the City of New York. While at the 'Soo', 1870-1882, Mr. Alfred Noble designed and built the Weitzel lock, St. Marys Falls Canal. It was a bold undertaking, for in lift and size it was a wide departure from any existing locks. Previous lifts in locks had been limited to about 10 ft., the 'Soo' lock provided for 20 ft. at extreme lift. It was 515 ft. long between hollow quoins, 80 ft. wide in the chamber narrowing to 60 ft. at the gates and with depth of 17 ft. of water on the miter-sills. The masonry was the finest of its kind ever built in this country. The filling and emptying culverts located under the floor of the lock, the gate hangings and the hydraulic operating machinery were all new features. The gate and valve engines have been in constant use every season since 1881, and have worked easily, efficiently, and rapidly, without any failure and without repairs except the annual repacking of the cylinders and occasional renewal of minor parts such as bolts and cables. Among other improvements to the canal was the deepening,

widening, and straightening of the old State Canal, the replacement of the paved side slopes with vertical walls of timber revetment, and the building of a movable dam consisting of a swing-bridge carrying a modified form of a Chanoine wicket, which was designed by Mr. Noble for a barrier against Lake Superior in case of wreckage of lock gates.

"The survey of St. Marys River, extending from Lake Superior to Lake Huron, a distance of 65 miles, was made in 1879, and the Lake George route was deepened from that of 12 to a navigable depth of 17 ft. and the narrow, tortuous channel was materially straightened and improved to a general width of 300 ft.

"Mr. Noble was in charge of operating the canal for a year and a half.

"The expenditures on the canal and river improvements made by Mr. Noble totaled about $3 000 000. His salary was gradually increased to the munificent sum of three thousand dollars a year. He resigned in August, 1882, to accept a position as Resident Engineer under Mr. Bouscaren on construction of the Shreveport, La., bridge at a salary of twenty-one hundred dollars a year.

"While at the 'Soo', Mr. Noble did the work of three or four expert engineers. He worked twelve to eighteen hours every day and his only vacations were taken during the last three years of his stay there when he spent about a week of each year trout fishing along the east shore of Lake Superior.

"He was a remarkably fine mathematician, a rapid and accurate computer, an expert draftsman and penman, and had the gift of writing concise, plain, and accurate statements and reports.

"He had the engineering sense to grasp the broad and controlling feature of a great work and the rare faculty of also grasping all the intricate details pertaining to the problem. Whenever a question was asked him about any part of the lock work, he could give at once a correct statement without stopping to think it over before being able to recall the particulars and their related bearings. When several hundred men were employed on the work, he knew and called every one by name, and could tell the value of each man as a workman. He was always pleased to find any employee taking special interest in his work, and would cheerfully aid in furthering that interest by explanation, by teaching, or in other ways. He was always pleasant, genial and sympathetic. He insisted on honest integrity, industry, clean and pure living in a man. He seldom spoke disparagingly of any person. I only knew of three men whom he personally disliked, and those three he believed to be dishonorable and hypocritical. He was generous. By accident I have learned of several instances where he has contributed considerable sums of money regularly for one or more years where employees have been injured, or who had dependent families sorely in need of assistance. I have known him to be tried by all kinds of aggravating conditions and subjected to most trying annoyances, but his wonderful patience mastered them all, only once have I seen him thoroughly angry, and then he showed it only by his silence, limiting his remarks to 'yes' or 'no' for three days. He was a great and most successful leader. It was no wonder that all his employees were loyal to him, willing to give the uttermost possible to acceptably serve him. No military officers could possibly obtain such service from men under their command."

Letter from Joseph Ripley.

"Replying to your request of May 25 for a short synopsis of Mr. Noble's activities at Sault Ste. Marie, I am enclosing herewith a statement relating to his work at that place, and some other information relating to him, leaving it to you to cull out such parts as you may desire to use.

"While Mr. Noble undoubtedly understood and knew of his marked ability, he never appeared to realize that he excelled or to assert it, but had a quiet, unassuming, reserved, and kindly personality which was most attractive.

"I first met Mr. Noble in 1872 and have had intimate acquaintance with him since 1876. Those of us who have been attached to him in the kindly feelings of long acquaintance counted him as one of the few really great men we have known, and believe that one of the grandest and most useful men of this country has gone from us.

"The lock Mr. Noble built at the 'Soo' was named for Godfrey Weitzel, who was the ablest and broadest of all the officers in the Corps of Army Engineers, and he always gave Mr. Noble full credit for his part of the work at the 'Soo'. (See Johnson's Encyclopedia; Article on St. Marys Falls canal, which was written by General Weitzel.)

"When the first boat, *The City of Cleveland,* was locked through to the Lake Superior level, the occasion was made quite an event, and about twenty engineer officers were present. Mr. Noble did not ride with the officers on the steamer, but stayed on the wall, watching locking operations. I heard Major (later General) Roberts, author of 'Roberts' Parlimentary Rules', congratulate General Weitzel on the completion of the greatest lock in the world, a work which would be a great personal honor and give renown to General Weitzel personally and, through him, be credited to the Engineer Corps and add much prestige to it. General Weitzel replied that 'Alfred Noble deserved all the credit for designing and building the lock'. Other officers present joined with Major Roberts in strenuously objecting to General Weitzel's statement saying that he (General Weitzel) was entitled to all the credit and honor for the success as he had the entire responsibility and would have had to have taken all the blame and discredit if there had been a failure. General Weitzel forcibly remarked that his risk or any one elses as to the failure part did not count for anything, because he had Alfred Noble for his Engineer.

"When General Sherman made a tour of the Western forts, Mr. Noble was directed by General Weitzel to meet the party on arrival at the 'Soo' and to show them about the lock work. Mr. Noble delegated his assistant, Mr. Davock, to meet General Sherman while he (Mr. Noble) went up to the head of the canal and stayed there all day, so as not to put himself at all forward in the presence of so notable a man.

"You are of course familiar with Mr. Noble's experience on bridge work in his practice as Consulting Engineer and also with the part he had to do with the movable dam located near the head of the Water Power Canal at the 'Soo' and the Remedial Works located across the head of the Rapids. His study and report on the hydraulic conditions

resultant in the change of the regimen of the river due to the construction of the Power and Ship Canals at the 'Soo' was a complete treatise on the intricate problems relating to river and lake reservoir hydraulics.

"Mr. Noble's one recreation was trout fishing, and every year since 1902 he has spent from two to six weeks in camp along the north shore of Lake Superior with a small party of old associates. Each outing trip greatly benefitted his health and the last time I saw him, on March 31, he planned the details of the anticipated trip in July.

"I suggest that the 'Western Society' or else the 'American Society of Civil Engineers' publish a memorial volume for Mr. Noble."

From A. Mackenzie, Hon. M. Am. Soc. C. E., Brig.-Gen., U. S. Army.

"Your letter of September 3d reached Washington during my absence in Europe and itself became something of a wanderer, taking some time to come into my hands.

"I grieve over Noble's death: though our lives lay apart for many of the last years of his life, memory and occasional happy meetings kept fully alive the strong bonds of friendship, which were established back in the Seventies, when we were first thrown together and worked side by side in Detroit and at the 'Soo'. None of his legion of friends found more pleasure than myself in watching Noble rise to the top round of his profession and at the same time win the hearts of all through his personality.

"So frankly honest was he, that his whole life was an open book from the time he quietly entered upon his chosen profession at the Sault Ste. Marie Canal—while still a student—up to and through his career as a world-known master.

"Not the least of his great virtues was his inherent modesty, which, as is known to many of his friends, threatened at an early day to draw him to a different line of engineering work from that to which he proved himself so perfectly adapted and in which he succeeded so grandly. Not long after entering on his engineering work he was offered a professorship, under conditions which led him to look upon the offer with favor, but his friends of those days—of which I am proud to have been one—felt that Noble was destined to become an active participant in the great construction work of his profession, to a much greater extent than his natural modesty permitted him to admit, and fortunately our counsels prevailed.

"No life's record brings to the individual or to the Engineering Profession more honor than that of Alfred Noble."

From Ralph Modjeski, M. Am. Soc. C. E.

"Although I had met Mr. Noble as early as 1887, I did not come into close contact with him until 1892. He was then resident engineer of the Memphis Bridge and I was one of his numerous assistants. It was then that I came to love the man for his great and unusual qualities. He always came to the office first and usually left last. No matter how some of us tried to be on the work ahead of him, we always found Mr. Noble there. His treatment of his subordinates was ex-

ceedingly kind without being lenient. Always ready to help with word of advice or to turn up his sleeves and join in the work if he saw he could help. No work was too trivial or too irksome for him. Nothing was neglected or passed over. His great accuracy and quickness of figures were proverbial.

"It was my good fortune to occupy with Mr. Noble the same office in the Monadnock Building in Chicago from 1900 until he was called away to New York on the Pennsylvania Tunnel work. In 1901 we formed a partnership under the firm name of Noble & Modjeski, and were given the contract for the engineering of the Mississippi River Bridge at Thebes, Illinois. Previous to that Mr. Noble was on the Deep Waterways Commission and later designed some regulating gates and other work for Sault Ste. Marie Power Company.

"During the constructing of the substructure, Mr. Noble and I took many trips to Thebes together. On one of those trips we had a drawing-room. As Mr. Noble was a very large and an older man than I, I insisted he should take the large bed and I slept on the narrow couch. On the return trip, however, Mr. Noble sneaked into the drawing-room very early and went to bed on the couch. I noticed the manœuvre too late, and no amount of persuasion or pleading could make him give up the couch for the larger bed. This is given as characteristic of the man.

"Another characteristic incident: When Mr. Noble was going to Panama, he asked me to keep his club dues paid, 'For', he said, 'I should not like to be posted as delinquent, and again I should dislike *not* to be posted if I deserved it'.

"Our partnership continued until the opening of the Thebes Bridge in May, 1905. As mentioned above, Mr. Noble moved to New York to take charge of the Pennsylvania Tunnels in 1902. Even after that date, he visited Thebes from time to time and aided me with his valuable advice in completing the work.

"He was most scrupulous and generous in money matters. Always ready and desirous to give more than he received, not only in money matters but in everything else.

"I know of no instance when Mr. Noble declined to see anybody who called on him, or to discuss with any one even the most trivial subjects. He never refused to give advice when asked for it, even on purely personal matters, and my experience has been that his advice was always good. He gave it very clearly, being apparently able to grasp the situation at once and his reasoning was always convincing. Yet when, on very rare occasions, he was mistaken, he never hesitated to admit it. Although always very busy he never made his callers feel it. On the contrary, he was always leisurely and kind when talking with them.

"When work was slack he studied or classified his engineering data and worked always. His knowledge of engineering matters was most thorough and was not confined to one branch of engineering only. Bridge work, water-power, harbors, canals, tunnels, railroads were, one might say, his specialties.

"He was a great man and a great engineer. When I think of an ideal to work up to, both as engineer and as man, Noble comes to my mind first of all."

From O. H. Ernst, M. Am. Soc. C. E., Brig.-Gen., U. S. A. (*Retired*)

"I first came into close association with Mr. Noble in 1899 when the Commission was created to examine and report upon all the routes for a ship canal across the Isthmus between North and South America. Besides Mr. Noble and myself the members of the Commission were Admiral J. G. Walker, U. S. Navy; Gen. Peter C. Hains, U. S. Army; Hon. S. S. Pasco, formerly U. S. Senator from Florida; George S. Morison, C. E.; William H. Burr, C. E.; Lewis M. Haupt, C. E., and Prof. Emory R. Johnson. The elaborate investigations which the Commission had to make involved long journies in Europe and Central America in which the members were brought into remarkably close personal intimacy. In the journey to Central America and over the Nicaragua and the Panama routes, they lived for several months as a single family, and had every opportunity for observing the personal as well as professional characteristics of each other. The public accommodations were rarely sufficient for a party as large as ours, and the opportunities for the display of selfishness or bad temper were constant. Throughout these expeditions Noble's equanimity never for a moment deserted him. His sweetness of disposition and generosity of temper endeared him to all.

"His professional work upon this Commission was of a very high order. With untiring industry he mastered the details of every branch of the investigation, and then with sound judgment and judicial temperament he reached conclusions which could not be shaken. Mr. Morison, himself one of the most eminent engineers in the U. S., said to me one day that Noble would be a good man to build the canal. This is a fair illustration of the esteem in which he was held by his colleagues on that Commission.

"My subsequent association with Mr. Noble, aside from the Panama Canal, related particularly to the hydraulics of the Great Lakes, and confirmed me in the conviction that, for the solution of any engineering problem involving long and careful analysis, he had no superior."

From William R. Day, Esq.

"I have your favor of the 24th ultimo, and am glad to know that the National Engineering Societies of this country have appointed a committee to prepare a suitable memorial to the late Alfred Noble.

"It was my privilege to be a classmate of his in the University of Michigan, where we graduated together in the class of 1870. I have met him from time to time since, and have known of the great career which he has had in his profession, and am glad to know that it is the opinion of his associates that he was among the first engineers of this country.

"I well remember when Alfred Noble came to the University of Michigan, where he entered the Sophomore class in 1867. He was somewhat older than the rest of us, and, in my opinion, far more able than any of us. He had had three years' experience in the army, and those who knew him there said that he had been a faithful and valiant soldier. I do not think any of his classmates ever heard him speak of his army career. He probably regarded it as merely a part of his duty, and not a thing to be talked about. Moreover, he was at all

times the most modest and retiring of men. Those of you who knew him, I think, will have marked this characteristic.

"I was with him on a number of occasions after he had become a famous engineer, and know that he was ever reluctant to have any exhibition of special honor to him, when, as everybody knew, he deserved it.

"As I say, he was older than the most of us, and I think his army experience had matured him at an earlier age than men usually reach a proper view of the responsibilities of life. In college, while he was always friendly, kind, and helpful, his time was given to the faithful pursuit of his studies when he was not employed, as he was at times in his college course, in government work.

"In his case, the boy was father to the man. He was modest, kindly, industrious, and capable, as boy and man. I need hardly say to you that he had particular aptitude for the science of engineering, and unusual skill in the higher mathematics. While he was easily, in my opinion, the first man in our class, I do not think there was any of his fellow-students who had the slightest feeling of envy or jealousy toward him. By common consent he was our intellectual leader. We all liked him, and the more we emulated his example and tried to reach his attainments in scholarship, the better it was for us.

"The last time I saw Alfred Noble, was at the great Michigan banquet in New York in 1911, when as a member of the New York committee he did very much to make that function the great success it was. With his great qualities and achievements, he had a gentle vein of humor that made him the most agreeable of companions. In person, as you know, he looked his part, and was a most attractive man. To have known him and had his friendship is one of the most pleasant and valued recollections of my life. I was much saddened to learn of his death, and, as I have said, I am glad to know that the profession which he honored is taking measures to provide a permanent record of his great career."

From James H. Brace, M. Am. Soc. C. E.

"While thoroughly appreciating all the benefits of several years' close connection with Mr. Noble in some of his later works, the writer likes best to think of a month's vacation spent with him in fishing and sailing or rowing along the solitary north shore of Lake Superior.

"The happiest years of Mr. Noble's life were doubtless those in which he was employed at and about Sault Ste. Marie, Michigan. When he first went to live there, the country was very much isolated, particularly in the winter season, as there was then no railroad nearer than Cheboygan on the Southern Peninsula. After navigation closed the only means of communication was overland either by sleighs or on snow shoes. Mr. Noble frequently had to make the trip overland to Detroit on Government business.

"In the summer he often found it necessary to make surveys along the beautiful shores of the St. Marys River and through the bush that even yet covers most of the back country. In those days he occasionally found time to make short fishing trips along the rugged shores of the great lake to the north. It was here, too, that he formed some of his

most intimate friendships, partly among the residents of the town, and partly with his associates.

"After he had made an assured success in his profession, Mr. Noble formed the habit of spending a month or six weeks every summer along the northeast shore of Lake Superior. A sort of informal club was composed of his old friends of earlier years. Among these were Chase Osborne, George Kemp, Judge Steer, and Joseph Ripley.

"A comfortable outfit composed of a sail boat, row boats, tents, etc., was gathered together at the 'Soo'. Mr. Noble spent much of his spare time from early spring planning for this trip.

"He took especial pleasure in a friendly rivalry with George Kemp in seeing who could secure, for a present to the other, the most novel or outlandish fly or other device supposed to be attractive to trout.

"Mr. Noble liked well to bring his friends of later years on these trips and one could not please him better than by genuinely enjoying the outing. The guides and cooks were obtained from among the Indians and half breeds living near the 'Soo'. Many of them were well known to Mr. Noble, and had formerly been employed under him on the construction of the Weitzel Lock.

"On this occasion the actual trip took about three weeks from the time of leaving Sault Ste. Marie to the return to Michipicoten Harbor. During this time the party was continually out of touch with civilization, as no mail, telegrams, or telephone messages could reach it. Here Mr. Noble took complete relaxation from his usual cares and duties, and this was practically the only time through the year that this was the case.

"It was by no means an idle time, however, for there was the early plunge in the cold water of the Lake, then breakfast and a prompt start for the day's business. If the party was to move, camp was broken at once. The day was fully occupied either with fishing or traveling.

"Mr. Noble took especial delight in properly rolling his blankets in the way he had learned to do in the Army. As fond as he was of fishing, nothing could induce him to go out when there was an ample supply for food for two or three days in advance.

"During the trip there were some trying experiences from wind and weather, but throughout these, as well as during the sunshine, Mr. Noble displayed the same kindly good humor and thoughtful consideration for others that characterized all his relations with his fellow-men.

"In the long twilight after supper, Mr. Noble could sometimes be induced to talk of his war experiences. He was very reluctant at all times to discuss this subject. He seemed to believe that it was every good citizen's duty to serve, then when the war was over, go about his regular business as though nothing had happened; that the country owed him nothing for his services, and that there was no good in keeping up the old spirit.

"The most vivid impression, however, was that of the earnestness of purpose that actuated both Mr. Noble and the members of his Company that was mainly recruited from the farming district adjacent to Detroit. There was apparently no glamour about it. They knew exactly what they were fighting for, believed in the right of their

cause, did their utmost in a humble way for its success, and most of the original Company were more than glad, when that cause had triumphed, to return to the ways of peace.

"Mr. Noble also gave some idea of his struggles for an education, both before leaving home and after the war was over.

"He commented quite freely on many of his associates, and although some of his experiences must have been unpleasant, he was always generous in his praise of their good points.

"In conclusion one could not come back from these few weeks of close association with him in this solitary region without feeling a lasting influence for good."

FROM J. WALDO SMITH, M. AM. SOC. C. E., CHIEF ENGINEER, BOARD OF WATER SUPPLY, NEW YORK CITY.

"I beg that you will excuse me for the delay in answering your letter of October 24, asking me to contribute information which might be of assistance to you and the other members of the committee in preparing a suitable biography of the late Alfred Noble, with particular reference to his work in connection with the Board of Water Supply.

"Mr. Noble accepted appointment as one of the Consulting Engineers of the Board of Water Supply in September, 1909, at the time when the Pennsylvania Railroad improvements in New York City were nearing completion. Previous to this, he had been repeatedly urged by Mr. Bensel, the President of the Board, and myself to investigate certain special conditions, but he advised us that he felt that all his time and energy belonged to the Pennsylvania Railroad, and refused to entertain any offers made to him. This was characteristic of the man—conscientious almost to a fault, always rendering a high order of service, and refusing to devote his energies to, or do, anything which would detract from his usefulness on the particular work which he had in hand, no matter how strong the financial inducement might be.

"At the time Mr. Noble's services were sought, some misgivings had arisen in the minds of the members of the Board as to the practicability and ultimate success of the tunnel underneath the Hudson River between Storm King and Breakneck Mountains, and a bridge crossing the river at that point was being considered somewhat seriously. The Commissioners all agreed that they were entirely satisfied to be guided by the advice and conclusion which Mr. Noble might reach after making a careful investigation of the whole subject. By reason of his long experience in the design and construction of large bridges, as well as his recent experience in connection with the Pennsylvania tunnels, and more particularly because of his sound judgment, exercised only after the most careful study of all the details and conditions, he was ideally fitted to undertake such a task. For myself, I was prepared to place absolute confidence in his findings, because through my intimate knowledge of his work here in New York, as well as my close association with him, through the late Charles L. Harrison, I had been very strongly impressed by the soundness of his judgment, the breadth of his knowledge of engineering matters and the care with which he pursued his investigations, and

was convinced that he would render an absolutely impartial judgment and not be swayed by prejudices or any political considerations. His report to the effect that every expedient should be exhausted before the deep pressure tunnel was given up practically settled all questions which had been raised.

"His advice was most valuable in connection with many of the details of the design and construction of parts of the work, particularly in connection with the large pressure tunnel (18 miles in length) running under the City of New York and the large dams at Ashokan and Kensico. He never rendered snap judgment on any problem or question. His advice was given only after painstaking consideration and careful study of all the details and conditions. He would never attempt to give advice on any matter for which he was not thoroughly fitted by experience. His attitude was always one of helpfulness, and he gave largely of his store of experience and skill without price to any one seeking information.

"If there was a wreck or failure, he did not condemn the whole structure; he sought to save what was good and would stand the test of sound engineering principles and design. He possessed a very keen intuition, and was not unmindful of practical considerations of business or even political conditions that surrounded any problem, but his findings were never tempered by prejudice. He was always constructive, always working for something better; he was never destructive.

"Mr. Noble was not demonstrative. He talked but little, but what he did say was always very much to the point. He was generous and kindly, and more considerate of others than he was of himself. He hated deceit and misrepresentation in every form. I remember a little incident which occurred about a year ago, shortly before the change in the City's administration. He told me he could see that there was to be a strong cry for economy in all the operations of the City, and that, whether it was advisable or not, strong pressure might be brought on our Board for the reduction of expenses, and so he suggested that, as he was the last of the three Consulting Engineers to be appointed, he would retire, in order that the others might not be disturbed. Neither the members of the Board nor I would listen to such a proposition, as we believed that his counsel was too valuable to lose at a time when the work was to be put under test, and his services might be very necessary. It is undoubtedly true that there was no other man in the Profession who was held in higher esteem or who was so generally liked and respected."

From Charles P. Light, Esq.

"It is very hard to write an appreciation of a man as modest as Mr. Noble was, this very trait having the effect of keeping one from saying a good many things that he might have otherwise given expression to. He endeared himself to all of us through the deep interest that he took in the work of our Association. The demands upon his time were multitudinous, yet he never missed a meeting of the Association. Mr. Saunders of the Ingersoll-Rand Company, gave voice to a sentiment concerning him that I most heartily concur in, it being as follows:

"'OBITUARY

"'Alfred Noble

"'At three score years and ten a useful life
Has run its course. And as we think of him
The Sorrow and the flowing tears of friends
Are turned to joy that such a one as he
Has lived and wrought. Here was a man who led
In building up, a mind endowed to see
And think and do in all the larger things,
A Captain leading men on Nature's fields
To win in building monuments of peace.
This Engineer has shattered Nature's works
To make the world a better dwelling place
For all of us. His life was gentle and
No thought of self within him dwelt. He won,
Scarce knowing why, the plaudits of the world.
Upon his monument let it be writ:
He was an Engineer. He was a Man.'"

FROM W. H. BURR, M. AM. SOC. C. E.

"My acquaintance with Mr. Noble began when he was engaged on the work of construction of the Washington Bridge across the Harlem River at New York City. This, however, was but a casual meeting on two or three occasions at most. My close acquaintance with him began only after he had completed the Memphis Bridge and had returned to Chicago to commence his independent consultation practice. I remember particularly meeting him in Chicago in December, 1896. We talked much about the foundations of the Memphis Bridge on which he had recently been engaged and for the success of which, it may properly be said, he was mainly responsible. It was not in accordance with his nature to make such a statement, but I write it as being just to him.

"He was probably one of the most modest men in the Profession, and never failed to accord to his chief all credit for the conception of work and the principal administration of it.

"The conception of the design of the Memphis Bridge, both superstructure and substructure was, of course, Mr. Morison's, and he was responsible for the general administration of the work, but I think it may be properly said, without in any way detracting from the credit due to the chief, that there were exigencies in the course of the substructure operations when Mr. Noble's presence and personal supervision and his fine engineering judgment were literally the saving of more than one threatening situation.

"At the period when he was engaged on this bridge he had reached mature age and had enjoyed abundant opportunity, through experience in many important works, to develop a well-trained judgment effective for the wide range of engineering operations for which he was noted during the last twenty or more years of his life.

"It was but three years later when the first Isthmian Canal Commission was appointed by President McKinley with Mr. Noble as

one of its members. His fitness for this Commission was greatly enhanced by the fact that he had already been a member of the Nicaragua Commission for the purpose of examining and reporting upon the Nicaragua route for a ship canal between the Atlantic and Pacific Oceans.

"The work of this first Isthmian Canal Commission extended over about a year and three-quarters, although it had official existence for about two years longer.

"It was as a colleague on this Commission that I came to know Mr. Noble most intimately. The work of the Commission was of a pioneer nature. Little was authoritatively known of the Nicaragua route and grossly exaggerated statements, to say the least, regarding the French work at Panama had greatly obscured knowledge of the Panama route. It was the duty of this Commission, therefore, not only to make the most thorough physical examinations of the Nicaragua and Panama routes, but also to visit Costa Rica and the Isthmus of Darien. Large engineering forces were at work for many months in both Nicaragua and Panama, securing data by surveys, borings, and other examinations which required extended treatment subsequently at Washington in the preparation of the report. In all this work in Central America and at Panama, and subsequently in the reduction and preparation of data in Washington, Mr. Noble was skillful, wise and tireless. He was not only the experienced professional man, but most gracious and invariably kindly in his relations with every member of the Commission. He was patient in times of difficulty, and frequently lightened the troubles of many unwelcome conditions by bits of quiet humor in which he was wont to indulge.

"He was one of the most companionable of men and while he could express himself with vigor whenever occasion might demand it, his nature was to accomplish his purposes through quiet and gentle procedures. In fact, he may properly be characterized as a gentleman in the best sense of the word.

"Although he would have been one of the last men to assert the possession of mathematical ability or of mathematical tastes, as a matter of fact, on a number of occasions, I have seen him exhibit greater power of mathematical analysis than is found among most engineers. It became necessary in preparing the report of the first Isthmian Canal Commission to consider the treatment of some hydraulic questions of magnitude and of much more than ordinary difficulty. Some of this work came under the scrutiny of Mr. Noble, and his treatment of the requisite analysis did credit to his mathematical ability.

"I saw the same quality exhibited in connection with some preliminary work for the Barge Canal of the State of New York while I was a member with him of a Board to which some questions connected with that work had been submitted. This analytic quality of Mr. Noble's mind, I think, has not often been recognized, even by many of those of his own profession who knew him well. It is of interest in connection with the deprecatory observations usually made regarding the possession of mathematical capacity by engineers. The possession of that faculty certainly did not trench seriously upon the excellence of Mr. Noble's engineering judgment.

"Throughout the whole laborious operations of the first Isthmian Canal Commission, Mr. Noble bore his full share from the beginning to the end, and his services aided much in giving to the report its high value.

"After becoming a member of the Isthmian Canal Commission he resided uninterruptedly in New York City, and my acquaintance with him was continually close until his death. During the last four years of his life we were associated on the Board of Consulting Engineers of the Board of Water Supply of New York City, Mr. John R. Freeman being the third member. This professional work included much deep tunneling for which Mr. Noble was finely equipped by his experience in the construction of the East River Pennsylvania tunnels and the tunnels connecting them with the Pennsylvania Station. This work, like all that he had done before, was characterized by great thoroughness. Whenever a problem arose his treatment of it was characterized by a patient thoroughness which could scarcely fail to lead to effective solution of any troubles which might be encountered. It seems to me that he possessed a capacity for deliberate and searching consideration of all the elements of engineering problems seldom equalled by any member of his profession. I have thought that this was largely due to the mathematical quality of his mind to which I have already alluded, but which seldom found expression by mathematical formulæ.

"He possessed unlimited stability and poise of mind. He could not be surprised into a conclusion not justified by his judgment, and it was unthinkable that he should reach an unwise conclusion through crude impulse. In endeavoring to find what qualities gave him the prominent position in the profession which he held, I think one must look chiefly to his perfect stability of character and judgment, his kindly nature unfailingly exhibited to all those with whom he came in contact, his uncompromising right principle, and his fine analytic capacity which he brought to bear on all engineering questions. He was not a man equipped with what may be called a brilliant searchlight of genius, challenging admiration by his phenomenal mentality and thus becoming an acknowledged leader of men in spite even of opposition. Alfred Noble was not of that type. He won his position of professional prominence by the more substantial and never failing qualities of personality and character and by his kindly good will, which always made him attractive. He was not a leader in the aggressive sense of the word, but the profession of which he was so long an honored member accorded him a high position because he had won it by the excellence and real worth of all that he did and was."

From Robert Ridgway, M. Am. Soc. C. E.

"Some time ago Mr. J. Waldo Smith showed me your letter to him of October 24, 1914, requesting him to contribute information to assist you and other members of the committee to prepare a biography of the late Alfred Noble, and suggesting that I might be able to contribute something as well. He has also given me a copy of his reply of December 10th, which is so complete regarding Mr. Noble's

connection with the Board of Water Supply work that I can add nothing to it.

"I presume you know that Mr. Noble was retained by the Public Service Commission for the First District, State of New York, on the recommendation of its Chief Engineer, Alfred Craven, M. Am. Soc. C. E., to act as consulting engineer to him. This appointment became effective November 1, 1912, and terminated with his death. He entered upon his duties with the conscientious thoroughness that was so characteristic of him, and his advice was a great assistance in solving some of the large engineering problems of subway design and construction in connection with the execution of the Dual System of Rapid Transit for New York City. Particularly is this true of his work on the specifications and features of design for the new East River tunnels and their connections which are a part of the Dual System. Each tunnel, or rather pair of tunnels, will consist of two single-track cast-iron-and-concrete-lined circular tubes of a type generally similar to the present subaqueous transportation tunnels about this City. The tubes for the Interborough Rapid Transit Company's system will extend from Whitehall Street, Manhattan, to Montague Street, Brooklyn; those for the New York Municipal Railway Corporation's system will run from Old Slip, Manhattan, to Clark Street, Brooklyn. I presume you have whatever details you may need of these tunnels, but if not, I will be pleased to furnish them if you desire me to do so. The engineering features of the contracts and specifications and the general designs were prepared under the direction of the Chief Engineer by Mr. Alfred Noble's son, Frederick C. Noble, then Engineer in charge of our Sixth Division, which included the East River Tunnels, and they were carefully reviewed by Mr. Alfred Noble, and in their final form are the result of his advice. His experience with the construction of the Pennsylvania Railroad tunnels under the East River gave added value to his advice. These contracts have since been let to the Flinn-O'Rourke Company, Inc., at the bid price of $12 444 725.

"You have known Mr. Noble so long and so well that anything I might say about his personal characteristics would simply confirm what you already know. He was considerate of the honest opinions of others, and was always ready to give full credit to his subordinates, including the laborers in the workings, for whatever of good they suggested or accomplished.

"He was intolerant, however, of incompetency and pretense.

"If there is any further information you desire that I can furnish, please command me. I am sorry this letter has been delayed, but when I read Mr. Smith's letter to you I was under the impression it gave you all the information that I could give you. It occurred to me only recently that perhaps what I have told you of his work with this Commission might be of interest to you."

From Henry Goldmark, M. Am. Soc. C. E.

"As suggested by you, I consulted with Mr. F. C. Noble with regard to the data desired in connection with the life of his father. There were a few dates as to which I was able to make Mr. Noble's

notes more complete. I do not know that I can add very much to such information as you already have.

"My own acquaintance with Mr. Noble dated back to the early 80's. At that time he had recently left the government employ and was with the Northern Pacific Railroad, engaged in active construction. I was inspecting the ironwork for a bridge at the south crossing of Clark's Forks, while he was in charge in the field. This bridge was designed by Mr. Geo. S. Morison. I was even then greatly struck with the manner in which Mr. Noble followed up every detail on this bridge. From 1888 to 1892, Mr. Noble was resident engineer for the Memphis Bridge, while I was stationed at Kansas City as Bridge Engineer for the Kansas City, Fort Scott and Memphis Railroad and allied lines. Mr. Geo. H. Nettleton, President of both Companies, and one of the finest men I ever met, often spoke to me about Mr. Noble, expressing his admiration for the latter's high qualities. The way in which he handled that big bridge was a revelation to me, especially his thoroughness and the total absence of friction in the organization. While the plans were made elsewhere, the successful completion of this difficult work was due very largely to Mr. Noble.

"It was not until 1897 that I worked directly under Mr. Noble, when he was one of the United States Board engaged on plans and surveys for a ship canal between the Great Lakes and New York Harbor.

"He was very anxious that the subject of lock gates should be thoroughly investigated from a broad, practical standpoint, and was quite willing that ample time should be spent upon this study. I am sure those engaged on it would never have had the perseverance to finish the laborious task except for Mr. Noble's own example.

"Apart from his very lovable personal traits, I have never met any one who, as an engineer, combined an infinite capacity for detail with the broad, common-sense view of the points involved in an engineering undertaking. While not without prejudices and strong opinions, he was always willing to discuss debatable points, and was readily convinced when the weight of the evidence called for a change of opinion. He chose his assistants with care, and required a great deal of them, although not as much as he demanded of himself. When he had once given a man his confidence, he was entirely willing to leave to him the carrying out of his instructions, and such suggestions as he made were always conveyed in a kind, generous manner, which made it a delight to talk over any point with him.

"Personally, I have felt his loss severely, and the world is to me poorer since the chance of meeting him from time to time has gone."

From Logan Waller Page, M. Am. Soc. C. E.

"In the death of Alfred Noble the American Highway Association has lost its greatest and most useful member. It was not alone through his eminent reputation as an engineer, or the mere lending of his name to the undertaking, that made the Association succeed. Before the founding of the Association was decided upon, Mr. Noble's sound advice and inspiration guided the few interested in the movement to direct their efforts along practical and useful lines. He

attended the founders' meeting, and was there elected a member of the Executive Committee, on which he served to the time of his death. During the four years that he served on this committee he never missed a meeting. The last meeting that he attended was in Detroit, Michigan. He made the long trip from New York City to Detroit and return for the sole purpose of attending this meeting, and at a real sacrifice to his private interests.

"When the Joint Congressional Committee on Roads, Congress of the United States, invited the Executive Committee of the Association to advise it in regard to Federal aid legislation, Mr. Noble attended the hearing and submitted to a long cross-questioning on the subject.

"These few examples of his generous and continued effort are given to illustrate the deep interest he took in work of a purely public service character. Not only did he give freely of his time and best judgment to the affairs of the Association, but he was always liberal in his financial support. He also drew many of his friends, who were among the most eminent men in the country to the councils of the Association. He was many times asked to accept the presidency of the Association, but, in his modest way, declined to accept any position of prominence, saying that he could serve the Association just as well on its Executive Committee as he could as its president. The loss of his counsel, advice, and deep interest has been the greatest blow the Association has sustained."

From Ralph Modjeski, M. Am. Soc. C. E.

"In answer to your recent reminder as to information regarding Mr. Noble's connection with the Thebes Bridge, the following may be of use.

"Mr. Noble and I formed a partnership for the purpose of engineering the Thebes Bridge over the Mississippi River at Thebes, Illinois, in November, 1901, at which time we were engaged jointly to design and build that bridge. From that time until January, 1905, Mr. Noble devoted a great deal of time to preliminary work on that bridge, and to designing of the substructure. There had been a tacit understanding between us that Mr. Noble would take care of the design of the substructure, leaving the superstructure largely to me. In January, 1905, he was called away to Galveston in connection with the new sea wall, and while there he received a proposition from the Pennsylvania Railroad to take charge of their East River Tunnel in New York. He did not wish to accept that proposition until he had ascertained that it would be acceptable to the railroads who were building the Thebes Bridge, and to myself, and until he received assurances from the Pennsylvania Railroad that he would be permitted to come to Thebes from time to time, to supervise the work in a general way. He finally made arrangements to that effect, and left for New York in February, 1902. He came to Thebes from time to time during the construction of the bridge, and devoted considerable time to making a final settlement with the contractors for the substructure.

"I feel that to Mr. Noble's wide experience and wisdom is largely due the success with which the work was carried on to its completion,

and it was mostly due to his great tact that the very complicated situation with the substructure contractor—due to delays caused by high water and other circumstances—was finally settled in a manner satisfactory to all concerned.

"The bridge was completed in April, 1905, at which time our partnership was automatically dissolved."

FROM HUGH L. COOPER, M. AM. SOC. C. E.

"American Engineers will be universally shocked as the news is conveyed to them of the death of Alfred Noble, a man who has been honored with the presidency of the American Society of Civil Engineers, and who in his lifetime achieved a far greater honor in the universal respect in which he was held by every one who knew him or knew of him.

"It was not my good fortune to know Mr. Noble personally, except in a very casual way, but his life work has been an inspiration and an exalted example to me for twenty-five years, as I have no doubt it has been to hundreds of other engineers.

"Growing out of this feeling, it has occurred to me that it is a very opportune time for the engineering profession in some unusual way to recognize the value of his services to the world at large, as well as to engineering. Can we not now inaugurate a strong movement having for its purpose the erection of some suitable monument or memorial illustrating in some degree to generations to come the great work Mr. Noble performed and the loss the nation suffers in his death?"

FROM JAMES FORGIE, M. AM. SOC. C. E.

"I am honored by a request from Mr. Modjeski, Chairman of the Biographical Committees of three engineering Societies, as a one-time Britisher representing the Institution of Civil Engineers (in America), to give a contribution in writing to the memory of Mr. Noble. It includes tributes by the following British Engineers—Mr. Charles M. Jacobs, Mr. E. W. Moir, Sir Maurice Fitzmaurice, and Mr. Henry Japp, who all have kindly permitted me to incorporate them in this memoir.

"Some people, maybe the weaker of us, are greatly influenced by the lives of others, in youth, maybe, by a biography such as the 'Lives of the Engineers' (Smiles); and again in youth and in manhood, by actual touch with the real lives of men. I must confess to this weakness, and among not a few Engineers of rare good character and technical ability it has been my good fortune to know, profit by their example, and work with, and which include those highest in the Engineering profession here and in Britain, there is none more dear than the late Mr. Alfred Noble. I was closely associated with him socially and often professionally for about twelve years and to me he embodied all the manly qualities.

"Any one who knew Mr. Noble is restrained from eulogy regarding him for two reasons: first, his character, to which obituary may do and usually does injustice, and, second, because of the disfavor he

would view our biographying him. For the common good of us all and future generations of engineers, this restraint must be laid aside.

"I have received from Mr. Chas. M. Jacobs, who was associated with Mr. Noble on the great Pennsylvania Railroad extension into New York City, his appreciation of the characteristics of Mr. Noble. It follows:

"'What do we live for if it is not to make lives less difficult for each other' (George Eliot).

"'That was the spirit of Alfred Noble in my personal experience during daily intercourse with him from January, 1902, to March, 1910, the time we were associated Chief Engineers, as well as Members of the Board of Engineers on the Extension of the Pennsylvania Railroad into New York City and Long Island.'

"'In all my experience, extending for a period of over 40 years, I have never been in contact with one so singularly independent and with such simplicity of character.'

"'One of the chief characteristics of his great professional attainments was the painstaking care which he devoted to the minutest detail of the subject under consideration, and his research on the many questions and new conditions that had to be dealt with.'

"'I have sat for hours, I may say days, with Mr. Noble, taking under consideration the multitudinous phases of the complex questions that were involved in order to reach a solution of the problems before us on the Pennsylvania work. He had to be absolutely convinced of the correctness of every detail before a final decision was reached.'

"'I can say here that, at our last meeting, the fact that during the entire period of our association, not a single word of anger or harsh criticism had passed between us was mutually a matter of sincere congratulations.'

"'As a man he was of the highest standard of honour and integrity, and was the very personification of humility. I can only add my testimony to the fact that the United States of America, and the profession generally, have lost one of the most distinguished Engineers of this generation.'

"To those, young and old, who knew Mr. Noble, the memory of his character and professional methods will remain fresh and helpful.

"Because of the character influence of Engineers of the past on the lives of following generations of engineers, one cannot but hope that by some means Alfred Noble's exemplary and vigorous life may be presented to future generations of engineers in such a way as to be helpful and encouraging and serve as a reminder that the 'right' can never be 'wrong', despite consequences of following one's conscientious judgment.

"Perhaps no engineer of foreign training was for so long a time continuously in touch with Mr. Noble, professionally and socially, as myself, with the result that, of the many blessings of fellowship enjoyed here, none has been of more moral value to me than this association with him.

"To illustrate his most sensitive fairness, permit the following: He consulted me, for not more than two hours at the most, on a

certain matter with which I happened to be also familiar and, to my surprise, sent me a check for half his fee, which, to satisfy his pride, I had finally to accept.

"He was an aristocrat of honor, but an autocrat toward those who, while knowing better, did not exercise it. Much may be said, and rightly, of his tolerance, helpfulness, and keen sense of humor, but it should not be forgotten that he in no sense overlooked a wrong.

"Mr. Noble was always an unstinting admirer of the oldest association of Engineers, 'The Institution of Civil Engineers', long before that body did him the honor of election as its Honorary Member in America. Those who have read the biography of Telford, the first President of the Institution, and who knew Mr. Noble will find a considerable similarity of character in these men. May I also say his brevity and pungency of speech recalled to my mind the manner and character of the simple and great British Engineer of our times, Sir Benjamin Baker.

"While Mr. Samuel Rea, President of the P. R. R. Co. proposed making the dinner of the Members of the Institution of Civil Engineers in America an annual event, it is to Mr. Noble we members owe the inception of the first one given in honor of Professor Unwin (then President of the Institution) at the University Club, New York City, on September 12th, 1912, during a visit to this country. This annual event affords as much pleasure to the Institution in London as it does to those who actually share in it.

"At the University Club on August 29, 1912, it was my good fortune to be the only foreign-born guest on the occasion of a dinner assemblage of Engineers given in honor of Mr. S. B. Williamson, Engineer in charge of the Pacific Division of the Panama Canal. This was five days after the passing, on the 24th, of the Panama Canal Act of 1912, in which exemption from tolls of coastwise vessels was among other matters enacted. In his address, at this dinner, and as first speaker, Mr. Noble, jealous of the honor of his country, stated in a most unqualified way that such exemption, no matter how desirable or undesirable it might be, was in direct contravention of the Hay-Pauncefote treaty. As every one knows, the honor and good sense of the country prevailed, and Mr. Noble lived to see this exemption part of the Act rescinded.

"Mr. E. W. Moir, who really won his spurs as an Engineer in this country on the old Hudson Tunnel in 1890, and who, as a Partner and Chief representative of the Contractors on the construction of the tunnels under the East River for the Pennsylvania Railroad Company, had to transact much business on this scheme with Mr. Noble, sends the following:

"'I duly received your latest appeal for some remarks on our mutual and dead friend Noble. I think I have already said that my admiration for him is very great indeed. I will make some effort to put something on paper that will be worthy of him. I am afraid, however, anything that I can say will not add to the high opinion his countrymen have already of him and of his works.

"'He was certainly one of the finest types of manhood that I ever met, either in the United States or anywhere else: able, kindly,

strong-minded, sticking to his opinions with great determination no matter how persuasive the arguments on the other side, and very thoughtful of others and generous in his dealings with them. I should say he was much the same type of character as Abraham Lincoln.

"'I spent a few enjoyable days in camp with him, sleeping in the same tent on the North Shore of Lake Superior; and perhaps one gets to know a man much more intimately if one practically shares the same bed in the wilds, than by months of association in a city like New York, with all its distractions and intensity of human endeavor.

"'We went through some very strenuous times together when we were building the East River Tunnels—a most trying job for the nerves—and while we had some differences of opinion on Engineering matters, we never differed enough to alter our mutual friendship in the slightest degree.'

"Sir Maurice Fitzmaurice, C. M. G., a Member of the Council of the Institution of Civil Engineers, sends the following tribute.

"'I only had the pleasure of knowing Mr. Noble for about five years and only met him on five or six occasions. I was always struck by his great sincerity and the extremely fair way in which he examined any questions put before him. I felt that I should be quite satisfied to take his opinion as an arbitrator on any question which might be in dispute in which I might be one of the parties. I say this not only on account of his professional qualities, which were as well recognized in Great Britain as in the United States and Canada, but also on account of his fair mind and common sense.

"'When a vacancy occurred among the Honorary Members of the Institution of Civil Engineers, some five years ago, I had the honor and pleasure of proposing Mr. Noble to fill the vacancy, and this nomination was unanimously accepted by the Council of the Institution and confirmed by the Members. We were very proud to have him as an Honorary Member, and only regretted that he filled that position for such a short time.'

"Mr. Henry Japp, Chief Engineer and Director for the Contractors of the P. R. R. East River tunnels, who submitted to the rulings of Mr. Noble on this work, sends what he calls his point of view; it is as follows:

"'Noble by name and noble by nature, like all great men, he was entirely unassuming, patient, painstaking and hard working; kindly, generous and unselfish; capable of meeting any obstacle and overcoming it; strong and reliable; courageous and never compromising with what he considered wrong.'

"Mr. Noble's technical missives were composed of the fewest possible words, and what was left unsaid was equally as forcible as the 'said', and now and again, but in consonance, contained a touch of humour between the lines.

"While the greatest factor in the preservation of, or criterion as to, the safety of investment is the character of management personnel, of no less value is the character of an advisory Engineer. It was Mr. Noble's wealth of simple, robust, honest character which made

him so valuable a technical advisor and a great asset to a great country. Such value has been concisely expressed:

"'We put too much faith in systems and look too little to men.' (Disraeli).

"'The worth of a State in the long run is the worth of the individuals comprising it.' (J. S. Mill.)

"Mr. Noble's value is amply demonstrated by his works, also by the affectionate admiration of Engineers and others. It was the greatest privilege to have known him as an unconscious example and a helpful friend."

FROM CHARLES WARREN HUNT, SECRETARY, AM. SOC. C. E.

"I am glad to pay, however inadequately, my tribute to the memory of Alfred Noble.

"My term of office as Secretary of the American Society of Civil Engineers began in January, 1895, and during that year Alfred Noble was elected one of its Directors. Five years later he became Vice-President (1900-1901), was elected President in 1903, and subsequently, as Past-President, served as a Member of the Board for six years (1904-1909), he was therefore a Member of the Board of Direction for ten years, the last nine of which were (with the exception of 1902) continuous.

"To sum up in a few words his influence in shaping the policy of the Society during that period, which was one in which a number of difficult situations arose, would be impossible; but it may be said that he gave to all his duties the benefit of his great capacity for detail, and that his broad and wise views, which were never expressed without careful and painstaking deliberation, and were always delivered in that modest, unassuming and convincing manner so characteristic of him, seldom failed to prevail. It seems to me that the key-note of Alfred Noble's nature, as shown in his attitude toward Society affairs as well as in all other relations of life was unselfishness.

"It was my good fortune, not only to have been thus associated officially with Alfred Noble, but to have shared with him several vacation trips, at times having been in camp with him alone for weeks, and this close contact gave me opportunity to form a correct estimate of his remarkable character.

"Of his professional ability, which was conspicuous, much will doubtless be written by others more familiar with the details of his work than I. The qualities in him that I like best to remember were his gentleness, genuineness, geniality, quiet humor, thorough sympathy with, and readiness to help others, by kindly advice or otherwise, wherever and whenever such help was asked or appeared to him to be needed.

"I feel that I owe much to him, and am proud to say that he was my friend.

"Doubtless there have been men of greater genius in some particular direction, men who were perhaps more deeply read, perhaps more broadly educated, men who, in their generation, have been more in the public eye; but as I am writing this I have tried to think of

all the attributes of which a man would wish to be possessed, and have endeavored without success to find one which was not a feature of his character.

"Alfred Noble was the best balanced, most lovable, most dependable, most useful man I have ever known. To meet him even casually was always a pleasure; to have known him intimately was a great privilege."

JAMES LEWIS FRAZIER, M. Am. Soc. C. E.*

DIED FEBRUARY 28TH, 1914.

James Lewis Frazier was born at Oakenwold, the fine estate of his father, at Staunton, Va., on June 17th, 1849. He was descended from the Scotch-Irish of the Shenandoah Valley, of Virginia, on both sides of the house. His father was William Frazier, of Staunton, an alumnus of Yale College and the University of Virginia, whose forbears came to Virginia from Ulster, Ireland. His mother was Sue Massie Lewis, of the sixth line of descendants from John Lewis, founder of Augusta County and Staunton, and his wife, Margaret Lynn Lewis, the daughter of the Laird of Loch Lynn in Scotland. John Lewis also came to Virginia from Ulster, Ireland; his sons, Gen. Andrew Lewis, Col. Charles Lewis, and Col. William Lewis, were conspicuous in Colonial and Revolutionary wars. Mr. Frazier was descended from Col. William Lewis (who was called the "Civilizer of the Border" during Colonial times), and his beautiful wife, Anne Montgomery.

His early education was under the direction of Dr. Junius, a French tutor at Oakenwold. Later, he went to Washington College, at Lexington, Va., and was there prepared for the University of Virginia under Gen. Robert E. Lee, then in charge of the college; he was a classmate there of Mr. Julius Kruttschnitt. He received his engineering education at the University of Virginia, and was a classmate of the late Samuel Spencer, M. Am. Soc. C. E.

Mr. Frazier was engaged on the location and construction of the Alabama Great Southern Railway, and the Chesapeake and Ohio Railway in the early Seventies, and was later associated with the late L. G. F. Bouscaren, M. Am. Soc. C. E., in the construction of the Cincinnati Southern Railway. He was also connected with the location and construction of many other lines of railroad.

In 1882, he was made General Superintendent of the Chesapeake and Ohio Southwestern Railway, with headquarters at Louisville, Ky. It was there that he married Miss Caroline Robinson, in 1887. In 1889, he was sent by the late Collis P. Huntington, F. Am. Soc. C. E., to the Southern Pacific Railway, and, under Vice-President Towne, was Superintendent, at various times, of most of the important divisions. At the request of one of the Directors of the Southern Pacific Railway, who was also interested in the Toledo, St. Louis and Western Railway, he became General Manager of that road for two years. He was then made General Manager of the California Northern, and returned to San Francisco.

*Memoir prepared by R. Montfort, M. Am. Soc. C. E.

As the result of an attack of pleurisy while in San Francisco, Mr. Frazier was left with serious heart trouble and retired from active work. He spent the last nine years of his life traveling with his wife, mainly in Europe. He died in Rome, Italy, on February 28th, 1914, after a short illness. His remains were interred at Louisville, Ky., the former home of his wife.

He had no children, but, being of a sunny disposition, was especially fond of them.

Mr. Frazier was an accomplished engineer and a born leader, endowed with all the energy, honesty, and courage of his long line of Virginia ancestors. His modest, lovable disposition endeared him to his many friends and associates who will ever remember him with affection.

Mr. Frazier was elected a Member of the American Society of Civil Engineers, on September 1st, 1880, and served as a Director from the Pacific Coast District from 1902 to 1904.

EMIL GERBER, M. Am. Soc. C. E.*

Died April 16th, 1914.

Emil Gerber was born in Reichenbach, Saxony, on January 31st, 1858, and died in Pittsburgh, Pa., on April 16th, 1914. His father, C. F. Gerber, was born in 1819 and died in 1899. He was a designer and manufacturer of textile fabrics, and introduced the use of power looms in his native town. His mother's maiden name was Christliebe Klotz, daughter of Carl Klotz. His ancestors for several hundred years were residents and prominent citizens of Reichenbach, Saxony. His father came to the United States in 1862 and settled in Webster, Mass., and in 1867 his mother came to Webster, bringing Emil and his two brothers, Herman and Carl.

He was educated in the common schools of Webster, Mass., and the Stevens High School at Claremont, N. H. In the fall of 1873 he entered the Worcester Polytechnic Institute, at Worcester, Mass., and was graduated as a civil engineer in the spring of 1876, being at that time only a little more than eighteen years of age. A classmate states that he was very much liked by all the students and professors. He was studious, a man of high character, and at that time gave promise to do good work. The President of the Institute writes: "Few students have done better in the Institute."

Mr. Gerber taught school for one year at Southbridge, Mass., and was employed as a bookkeeper in a corporation store connected with the woolen mills in Webster, Mass., until May 1st, 1879, when he was engaged as a Transitman on the Fremont, Elkhorn, and Missouri Valley Railroad. In May, 1880, he became Locating Engineer with the same railroad, and in 1881 was promoted to the position of Assistant to the Chief Engineer, Captain James Edward Ainsworth. On August 1st, 1881, he was appointed Assistant Engineer on the Blair Bridge over the Missouri River, near Blair, Nebr., built by the Sioux City and Pacific Railroad which later became a part of the Chicago and Northwestern Railway. The late George S. Morison, Past-President, Am. Soc. C. E., was Chief Engineer of this bridge. Toward the completion of this work, on November 1st, 1883, Mr. Gerber was made Resident Engineer.

In October, 1884, he returned to his former position with the Fremont, Elkhorn, and Missouri Valley Railroad, but still retained charge of the Blair Bridge and its extensive protection works. In November, 1885, he was appointed Assistant Chief Engineer of the same railroad, which position he held until he resigned on June 15th, 1887, to become Resident Engineer of the Sioux City Bridge

* Memoir prepared by Otis E. Hovey and August Ziesing. Members, Am. Soc. C. E.

E. Gerber

over the Missouri River, of which Mr. Morison was also Chief Engineer. This bridge was completed at the end of 1888, and Mr. Gerber was then appointed Resident Engineer of the bridge over the St. Johns River at Jacksonville, Fla., which was being built by the Jacksonville, St. Augustine, and Halifax River Railroad, with Mr. Morison as Chief Engineer. A severe illness prevented Mr. Gerber from completing this bridge, and in May, 1889, Mr. Morison placed him in charge of his Chicago office, where he remained until the fall of 1897.

During this period, he was connected with all Mr. Morison's important works, among which may be mentioned the Cairo, Memphis, Burlington, Winona, Bellefontaine, Alton, and Leavenworth Bridges; the Chicago, Burlington, and Quincy entrance into St. Louis, and many less important works.

In 1897, Mr. Gerber resigned his position with Mr. Morison to become Chief Engineer of the Lassig Bridge and Iron Works, in Chicago, Ill. He held this position until the spring of 1900. When the American Bridge Company was formed, he was made Manager of the Lassig Plant. In July, 1901, he was promoted, becoming Assistant to the President of the American Bridge Company, in Philadelphia, Pa. Early in 1904, he went to Pittsburgh, in the same position, and in the fall of 1905 was made Operating Manager of the Pittsburgh Division in addition to his position as Assistant to the President. In 1911, he was relieved as Operating Manager and made General Manager of Erection, but still held the title of Assistant to the President. He held this position during the remainder of his life.

He was married on January 3d, 1882, at West Roxbury, Mass., to Caroline Herthel, daughter of F. J. Herthel, Sr., and is survived by his widow, a daughter, Mrs. Laura E. Olson, of Chicago, Ill., and a son, Emil Gerber, Jr., of Pittsburgh, Pa.

As is often the case with engineers fully and continuously engaged on important work, Mr. Gerber was not a frequent contributor to engineering literature, but whatever he wrote is strongly marked by his vigorous personality.

He was the author of a paper entitled "Painting of Iron Structures Exposed to Weather",* and also of one entitled "Some Commercial Features of Structural Engineering".† He had based a few lectures to young engineers on the latter paper, and his last brief illness and death prevented him from lecturing on the same subject before the American Bridge Engineering Club of New York. This paper is one which should be read by all engineers.

It frequently happens that an engineer employed by individuals or corporations does not become so well and favorably known as he would have been had he been in practice under his own name. It is

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* *Transactions*, Am. Soc. C. E., Vol. XXXIII, p. 485.

† *Proceedings*, Eng. Soc. of Western Pennsylvania, Vol. XXIII, p. 125.

also common for an engineer associated with corporations to specialize along some narrow line of engineering work. Mr. Gerber was an exception in these respects. His early experience covered a very wide range of work, including railroads, difficult foundations, construction of bridges and other structures pertaining to railroads, and, at the time he took up his work as an engineer associated with a corporation, his experience was wide and his judgment well trained.

Those closely associated with him were impressed by the thorough manner in which he carried out any work on which he was engaged, his genial and helpful disposition, and his hearty and liberal endeavors to assist those around him in every way. He was a man who would ungrudgingly draw from his experience anything that would help his associates to grasp an engineering or business situation with which he was familiar.

He never did anything superficially. Any problem that was before him received his complete and earnest attention. Whenever he undertook an investigation or made a report, one could depend on his having gone into the matter thoroughly and having expressed his best opinion; and any facts contributed to the subject could be depended upon.

Mr. Gerber had a strong mentality and firm opinions on any subject to which he had given thought, but at the same time was always willing to discuss exhaustively the opinions of others, and when they were backed by facts and good reasoning was ready to modify his previous views in an eminently fair-minded manner.

A notable trait of his personality was his unusual ability in the establishment of cordial relations with his associates, and his interest in all that contributed to their welfare.

On account of his connection with a multitude of engineering works to which he contributed in more or less degree, and in which his strong common sense and clear thinking determined important decisions concerning designs and methods of construction, it is unfortunate that the Profession cannot point definitely to this or that work as purely his own. His monument is a more personal one. His strong, honest, thorough, and genial personality has impressed itself on all who knew him, and most of all on those who knew him intimately. Such characters are an inspiration to all members of the Profession.

Mr. Gerber was elected a Member of the American Society of Civil Engineers, on February 1st, 1888, and served as a Director from January, 1912, until the time of his death. He was also a member of the American Railway Engineering Association, the Western Society of Engineers (Past Treasurer), the Engineers' Society of Western Pennsylvania, the Chicago Engineers Club, and the Duquesne Club and the Junta Club, both of Pittsburgh.

JUSTUS HERBERT GRANT, M. Am. Soc. C. E.*

Died August 1st, 1914.

Justus Herbert Grant was born in Auburn, N. Y., on June 19th, 1849. His first engineering experience was obtained in 1866 and 1867, as Rodman on the preliminary and location surveys for the Southern Central Railroad.

In 1867, he entered the Sheffield Scientific School of Yale University, from which he was graduated in the class of 1870 with the degree of Ph. B.

In 1871, he served as Levelman and Topographer on the surveys for the Utica, Chenango and Cortland Railroad, from Cortland to Otselic, N. Y., and during a part of 1872, as Levelman and Assistant Engineer in charge of the construction of a short, narrow-gauge line, in Chenango County, N. Y., known as the Central Valley Railroad.

In 1872 and 1873, Mr. Grant was employed as Levelman on the preliminary surveys for the Auburn and Homer Midland, and Canandaigua, Palmyra and Ontario Railroads.

From 1874 to 1876, he was a member of the Engineer Corps of the New York Central and Hudson River Railroad, serving as Assistant Engineer in charge of earthwork and masonry construction of 16 miles and of the track-laying and ballasting of 32 miles of the line between Syracuse and Rochester, N. Y.

In the Fall of 1876, he resigned to accept a position as Engineer with the firm of George H. Thompson and Company, Contractors, of Rochester, N. Y., and as such was in charge of laying out and superintending the construction of all the work of that firm. In 1885, on the death of Mr. G. H. Thompson, the firm was dissolved, and Mr. Grant associated himself with the late H. M. Ellsworth, as the firm of Ellsworth and Grant, Contractors. This firm built many of the largest buildings in the city, including the Chamber of Commerce, the new Central Church, the Andrew Street Bridge, and the West High School.

In 1900 and 1901, Mr. Grant served as Commissioner of Public Works of Rochester, and in 1904 he was appointed Assistant City Engineer, in which capacity he served the city until his death on August 1st, 1914.

Mr. Grant was one of the most solid and influential business men of Rochester, of which city he had been a resident since 1874. His record and reputation in his profession were of the highest order, and he was regarded by his associates as one of the distinguished engineers of the United States.

* Memoir prepared by Charles H. Grant, Esq., supplemented by information on file at the Society House.

He was a Trustee of the Mechanics Institute, and one of the founders of the Rochester Builders' Exchange, having served as its Secretary for many years. He was also a Director of the National Association of Builders; a member of the Permanent Arbitration Board of the Builders' Exchange; a member of the Brick Layers', Plasterers', and Stone Masons' Union; a Trustee of the Rochester Home for the Friendless; a Trustee of the Unitarian Church; and a member of the University Club of Rochester.

Although the community apparently has lost a devoted servant, a citizen of public spirit and of pride in its progress and advancement, his presence will be always felt, his exemplary life will be always an inspiration to those who knew him and loved him, and his work will go on.

Mr. Grant was elected a Member of the American Society of Civil Engineers, on March 2d, 1892.

ROBERT MAXSON GREENE, M. Am. Soc. C. E.*

DIED DECEMBER 5TH, 1914.

Robert Maxson Greene, the son of David Maxson and Emma MacAlpine Greene, was born in Hillsdale, N. Y., on March 30th, 1870, and received his early education in the public schools of that place and elsewhere.

Prior to entering Rensselaer Polytechnic Institute, where he studied engineering for three years, until June, 1892, he had been employed, from July, 1887, to January, 1889, as Roadmaster's Assistant in the Engineering Department of the Missouri Pacific Railway, at Pueblo, Colo., and Scott City, Kans.

Mr. Greene had always been interested in structural and architectural work, and in September, 1892, he was appointed a Draftsman in the Structural Department of the Pencoyd Iron Works, Philadelphia, Pa., where he remained until January, 1894. From January to May, 1894, he was employed as Architect by the New Jersey Building and Supply Company, and from May to December, 1894, as Draftsman and Checker by the Jackson Architectural Iron Works of New York City.

In December, 1894, Mr. Greene went to Pittsburgh, Pa., as Draftsman with the Pittsburgh Bridge Company. He remained with this Company until July, 1895, when he returned to New York City and was engaged with Milliken Brothers as Draftsman and Assistant Engineer until November, 1898. During this time his work consisted of detailing and checking the structural steel and iron work for office and mill buildings, elevated railways, bridges, viaducts, etc., and, among other work, he had charge of the structural details for the following: Museum of Arts and Sciences and Polhemus Memorial Clinic Buildings, Brooklyn, N. Y.; the tower of St. Luke's Hospital and repairs to the United States Appraisers Warehouses, in New York City; six-story department store, in Scranton, Pa.; Leary Building, Seattle, Wash.; Terminal Station of the Chicago and North Western Railway, Chicago, Ill.; Hearst Building, San Francisco, Cal.; Lafayette Hotel, Buffalo, N. Y.; fifteen-story railway building in Buenos Aires, Argentine Republic; sugar factory buildings in the Philippine Islands; etc., etc.

From November, 1898, to January, 1899, Mr. Greene was employed as Designing Engineer by De Lemos and Cordes, Architects, of New York City. In January, 1899, he became a member of the firm of Foster and Greene, Architectural Engineers, of New York City, and on the dissolution of the partnership, in October, 1900, the firm was continued as Greene and Ward, Engineers and Contractors.

* Memoir prepared by George E. Thackray, M. Am. Soc. C. E.

These firms had charge of designing, detailing, and furnishing the structural steel and iron work for the Pugh Building, the Stewart Apartment House, and various other apartment houses and private houses, public schools, the First Battery Armory, Holy Trinity Church, etc., in New York City; the Roman Catholic Church, at Corona, Long Island; the Winter Club House, at Tuxedo Park, N. Y., etc., etc.

In November, 1901, Mr. Greene retired from the firm of Greene and Ward to engage in private practice, in New York City, as a Designing and Consulting Engineer. In this capacity, he had charge of the detailing, purchase and erection of the structural and ornamental steel and iron work for various buildings for the Sisters of St. Joseph at Brentwood, Long Island, a barracks for the United States Marine Corps, at Washington, D. C., and many other structures.

In September, 1903, Mr. Greene accepted a position with the Cambria Steel Company, of Johnstown, Pa., as Designer and Estimator on structural steel and iron work for bridges, buildings, viaducts, tunnels, etc.; he remained in this position until October, 1906. From December, 1906, until 1912, he was in the employ of the American Bridge Company of Ambridge, Pa., as Assistant Engineer in charge of detail construction of steel and iron work for office and mill buildings, hotels, etc.

From 1912 until his death on December 5th, 1914, Mr. Greene was engaged in private practice as a Structural Engineer, first in Detroit, Mich., and later in Winnipeg, Man., Canada. He had been in ill-health for the last few years of his life, but died suddenly while in Chicago, Ill., from hemorrhage and heart failure. He was married, on November 18th, 1896, at Manhasset, Long Island, to Miss Harriette Simpson Horsfield, who survives him.

He was a most pleasant gentleman, always thoughtful of the welfare of his companions and associates, and his early and sudden death will be regretted by his numerous friends throughout the United States.

Mr. Greene was elected an Associate Member of the American Society of Civil Engineers on June 1st, 1898, and a Member on September 1st, 1908.

JOHN CHARLES WILLIAM GRETH, M. Am. Soc. C. E.*

Died August 7th, 1915.

John Charles William Greth, the son of William and Caroline Greth, was born in Buffalo, N. Y., on May 15th, 1874. He attended the public schools of that city and was graduated from the Central High School in 1893. In the same year he entered Cornell University, from which he was graduated in 1897 with the degree of Mechanical Engineer.

After his graduation, Mr. Greth began his engineering work by installing, and operating for a few months, a power-plant at one of the summer resorts on Lake Erie. During the succeeding years, from 1898 until 1902, he successively installed pumping machinery, designed special machinery, operated a power-plant, and installed and operated refrigerating and ice-making plants.

In February, 1902, he entered the service of William B. Scaife and Sons Company, of Pittsburgh, Pa., as Manager of the department devoted to the softening and purification of water. From that time until his death his whole energy was devoted to the development of apparatus and methods for the softening and purification of water for all purposes. He was granted sixteen patents on improvements in water-purifying apparatus, several of which marked radical steps forward in the development of mechanical methods in water purification. In this field Mr. Greth occupied a dominant position, and his forceful presentation of the subject contributed very materially to the advancement of the science and to the broadening of its application for various industrial uses.

He contributed articles on that subject to the technical publications, and read a number of papers before various engineering societies. He was recognized as an expert in this art by engineers and chemists, and his sterling integrity gained for him a wide circle of friends and admirers. Mr. Greth was noted for his frank and friendly disposition, and numbered a host of friends among his acquaintances. By his sincerity and forceful personality, coupled with marked ability in his field of endeavor, he commanded the respect of all with whom he came in contact.

He had not been in robust health for several years, and the shock and grief over the loss of a two-year old daughter on July 14th brought on a nervous break-down, from which he did not have the strength to recuperate, and, after an illness of two weeks, he died on August 7th,

* Memoir prepared by M. F. Newman, Esq., Mgr., Water Purifying Dept., Wm. B. Scaife and Sons Co., Pittsburgh, Pa.

1915, at his summer home at Gibsonia, Pa. His remains were buried in Allegheny Cemetery, Pittsburgh, Pa.

In 1902, Mr. Greth was married to Miss Laura Heussy, of Buffalo, N. Y., who, with two sons, aged 11 and 8 years, survives him.

He was a member of the American Society of Mechanical Engineers, the American Institute of Chemical Engineers, the Engineers' Society of Western Pennsylvania, the American Chemical Society, Pittsburgh Commandery No. 1, Knights Templar, Syria Temple, A. A. O. M. S., and the Duquesne Club.

Mr. Greth was elected a Member of the American Society of Civil Engineers on March 4th, 1913.

FRANCIS HENRY HAMBLETON, M. Am. Soc. C. E.*

Died March 19th, 1915.

Francis Henry Hambleton was born on December 28th, 1834. When he was 20 years old, he entered the Winans' Locomotive Shops, in Baltimore, Md., where the first locomotive used on the Baltimore and Ohio Railroad, as well as the famous "camel-back" engines, used on the same road for many years, were built. Mr. Hambleton remained in the shops for three years and was then transferred to the drafting-rooms, where he remained for one year. He was afterward sent to Russia by the firm, to assist in railroad construction, and remained in that country for several years.

Mr. Hambleton spent a year making a series of experiments on the friction and resistance of bodies passing through water. He also had charge, for the Winans, of the construction of the steamer *Winans,* in Baltimore, and of the erection of its machinery, together with the numerous experiments made with that vessel. Later, he was sent to England, where he had charge of the construction of another steamer, the *Ross Winans,* and of the working out of data accumulated during several years of experiments with that boat. In appearance, these boats were somewhat like the torpedoes used by modern submarines, and were known as "cigar ships". They were designed by the Ross Winans Company to lessen the time of crossing the Atlantic Ocean. While in England, Mr. Hambleton was also occupied in experiments in the mechanical puddling of iron.

On his return to the United States, he was engaged in the design of water-works, and, for a time, was employed as Resident Engineer on the Norfolk City Water-Works. He was also engaged in sanitary engineering work.

In 1877, Mr. Hambleton began his long career in the gas industry as Consulting Engineer for M. S. Frost and Son, of Baltimore, Md. This firm had a contract for laying about 50 miles of distributing mains for the Consumers Mutual Gas Light Company, and Mr. Hambleton designed and had charge of laying these mains, which were of large size for that day, the largest being 30, 24, and 20 in. in diameter, respectively. After the distribution system had been completed, he was put in charge of the construction of the manufacturing plant of the Consumers Company at Canton, from which station the first water gas was distributed to consumers in Baltimore, on

* Memoir prepared by the Secretary from information furnished by George Beadenkopf, Chf. Engr., Consolidated Gas, Electric Light and Power Co., Baltimore, Md., and from papers on file at the Society House.

January 28th, 1878. The Canton plant continued in operation until 1904 when it was closed as a manufacturing station.

In 1880, the Consumers Mutual Gas Light Company and the Gas Light Company of Baltimore were consolidated under the name of the Consolidated Gas, Electric Light and Power Company of Baltimore, and Mr. Hambleton was made Engineer of the new Company. In 1887 another company, the Chesapeake Gas Company, was added to the system, and Mr. Hambleton was retained as Engineer of the combined companies until the latter part of 1902, when he was appointed Consulting Engineer. He remained in this position until 1910 when he retired from active service. He retained his membership on the Board of Directors, however, until his death, having served as a Director of the Consolidated Company for 12 years.

During the last 38 years of his life Mr. Hambleton had been identified with the manufacture of gas in Baltimore, but, he also took an active interest in the construction of the cable railroad on the various car lines of that city when rapid transit was being introduced.

Mr. Hambleton enjoyed the reputation of being an Engineer of varied and unusual attainments, his experience having been very wide and covering many fields. He was held in high esteem by his fellow-directors and his associates, and enjoyed the confidence of all who knew him.

The following is quoted from the minutes adopted by the Executive Committee of the Board of Directors of the Consolidated Gas, Electric Light and Power Company of Baltimore, on the announcement of his death:

"For more than a quarter of a century the Company and the people of Baltimore benefited by the technical knowledge and engineering skill of Mr. Hambleton, who as Chief Engineer of The Consolidated Gas Company and later as the Consulting Engineer of this Company, brought to the solution of important engineering problems signal ability as an engineer; and whose breadth of view and public spirit characterized him as one of the best informed and most useful citizens of Baltimore. For many years a Director of this Company, the members of the Board, whose privilege it was to be associated with him, and the employees of the Company who knew Mr. Hambleton, will always retain and enjoy the enduring and inspiring impressions of his honesty of thought and methods of attainment."

Mr. Hambleton was elected a Member of the American Society of Civil Engineers on March 5th, 1873.

WILLIAM MACKENZIE HUGHES, M. Am. Soc. C. E.*

Died June 25th, 1915.

William Mackenzie Hughes was born in Utica, N. Y., on June 5th, 1848. He began his career as a machinist's apprentice and worked at that trade until September, 1869, when he entered Cornell University. He left college, however, in February, 1871, without taking his degree.

In 1871 and 1872, Mr. Hughes was employed as Assistant to Marvin Porter, Chief Engineer of the Walnut Hills Tunnel, at Cincinnati, Ohio, and during the spring and summer of 1873, he was engaged as an Architectural Draftsman and also as Assistant Engineer on the survey of the Big Sandy Valley Railroad, under Mr. Porter, as Chief Engineer.

Mr. Hughes had chosen to make bridge and structural steelwork his specialty in engineering, and in September, 1873, he entered the service of the Cincinnati Bridge Company, as Assistant on the design and erection of bridges. He remained with that Company until 1876, when he was employed on bridge construction by Mr. Charles Graham, of Cincinnati.

In October, 1876, he was appointed Assistant City Engineer of the City of Cincinnati, in charge of bridges and bridge construction, which position he held until 1881, when he entered the service of the New York, Chicago and St. Louis Railway Company.

From 1883 to 1888, Mr. Hughes served as Bridge Engineer of the City of Cleveland, Ohio, and during 1889 and 1890, he was engaged as Engineer and Assistant General Manager with the Keystone Bridge Company. He left this Company in February, 1891, to become Engineer of Construction at the Columbian Exposition, under the late Abraham Gottlieb, M. Am. Soc. C. E., but resigned that position in August of the same year.

During 1893 and 1894, Mr. Hughes was Bridge Engineer of the City of Chicago, and from 1892 to 1896, during the construction of the Metropolitan (West Side) Elevated Railroad, he served that Company as Consulting Engineer. In 1896, he was appointed Bridge Engineer of the Sanitary District of Chicago, and remained in that position until May 1st, 1898, when he resigned to engage in private practice as Consulting Engineer, continuing in that capacity until his death. He is survived by his widow and one daughter.

* Memoir prepared by the Secretary from information on file at the House of the Society.

Those who knew Mr. Hughes prized his friendship and appreciated the force of character back of his quiet, unobtrusive manner. By his death, the Engineering Profession has lost a useful member and his associates a valued friend.

Mr. Hughes was a member of the Engineers' Club of Chicago, and was elected a Member of the American Society of Civil Engineers on June 2d, 1880.

HURD CLARENCE HURD, M. Am. Soc. C. E.*

DIED JULY 8TH, 1914.

Hurd Clarence Hurd was born at Helena, Tex., on April 16th, 1870. The early years of his life were spent in a community in which the Spanish language was spoken to a considerable extent. He acquired a knowledge of this language at that time and afterward perfected it in high school and university, making much use of it throughout his professional career.

He was graduated with the degree of C. E., from Princeton University, in June, 1893. His first work was in connection with the Sewer Department of Washington, D. C. In 1897, he went with the Nicaraguan and Isthmian Canal Commissions as Instrumentman, but at the end of the first year of his engagement he became First Assistant to the Chief Hydrographer, holding this position until 1901. Although this work was interesting, and developed individual initiative, it was also fraught with hardships, and Mr. Hurd, besides experiencing the minor troubles, also suffered an attack of yellow fever. That he came out of this country not completely broken in health, was a strong testimonial of his rugged constitution and temperate manner of living.

In September, 1902, he was appointed a Special Agent of the State Department, in Nicaragua and Costa Rica. In 1903, he became attached to the United States Reclamation Service, and spent the next two years in charge of important surveys and construction of canals, dams, and irrigation systems in Arizona, California, New Mexico, and Montana.

In 1905, the call of the South again reached him, and he went to Peru as Chief of the Hydraulic Commission for the study of irrigation. The Peruvian Government having requested the recommendation of a suitable person for this position, Mr. Hurd was named by the United States Government.

In 1907 Mr. Hurd returned to the United States and took up hydro-electric work in California. He followed this line of work for five years and, during that time, had charge of the construction of several important hydro-electric developments. His skill in reconnoissance surveys and plans gave him such a standing that he was employed, in 1912, by the Ebro Irrigation and Power Company, to take charge of the location, design, and construction of a large lined power canal, aqueducts, siphons, etc., in Spain.

He was in charge of this work for about one year when the general financial depression compelled its cessation. Mr. Hurd then returned

* Memoir prepared by Wendell M. Reed, M. Am. Soc. C. E.

to the United States and was taking a much needed rest when he accidentally met his death by drowning in Chesapeake Bay.

Mr. Hurd was modest and unassuming in manner, but soon impressed those with whom he came in contact with his sterling worth, and professionally he ranked very high.

He was married in 1905 to Miss Grace Lewis, of Roswell, N. Mex., who, with four children, survives him.

Mr. Hurd was elected a Member of the American Society of Civil Engineers on July 2d, 1913.

JOHN WYKEHAM JACOMB-HOOD, M. Am. Soc. C. E.*

DIED MARCH 6TH, 1914.

John Wykeham Jacomb-Hood was born at Lewisham, Kent, England, on January 12th, 1859. He was the second son of the late Robert Jacomb-Hood who, for many years, was the Engineer, and later a Director, of the London, Brighton, and South Coast Railway.

From 1870 to 1875, he attended Tonbridge School. He received his technical training at the School of Practical Engineering at the Crystal Palace.

In 1876, he was articled to the late William Jacomb-Hood, Chief Engineer of the London and South Western Railway, and from 1877 to 1887, was engaged in various offices of the Company on maintenance and construction.

In 1887, Mr. Jacomb-Hood was appointed District Engineer in charge of the London District, remaining in this position until 1896 when he was transferred, as Engineer, to the Western District with headquarters at Exeter, England.

In November, 1900, he was appointed Chief Resident Engineer of the London and South Western Railway, which position he held at the time of his death.

He died suddenly, from heart failure, on March 6th, 1914, at Dulverton, Somerset, England.

In his position as Chief Resident Engineer, Mr. Jacomb-Hood was responsible for many works of great magnitude, including the Woking to Basingstoke Widening; the extension of the road to Bulford Camp; the Bentley and Bordon Railway; the Axminster and Lyme Regis Railway; the new locomotive depot at Eastleigh; the introduction of power and automatic signaling on various parts of the System; the reconstruction of the bridges over the Thames at Richmond and Kingston; the construction of the stations at Salisbury, Basingstoke, and Clapham Junction; and the reconstruction of Waterloo Station which had not been completed.

Mr. Jacomb-Hood was a Member of the Institution of Civil Engineers and also of the Institution of Electrical Engineers. In 1911, he was President of the Permanent Way Institution. He is survived by his widow and a daughter, Miss Molly Jacomb-Hood.

He was a man of marked personality and of such a genial nature that he will be greatly missed. As Chief Resident Engineer, he was always greatly interested in, and accessible to, the members of his staff and was greatly beloved by all who knew him.

Mr. Jacomb-Hood was elected a Member of the American Society of Civil Engineers on February 5th, 1902.

* Memoir prepared by the Secretary from information on file at the Society House.

CHAPMAN LOVE JOHNSON, M. Am. Soc. C. E.*

Died March 11th, 1915.

Chapman Love Johnson, the son of Dr. Carter Page Johnson (a noted physician in his day and Principal of the Medical College of Richmond) and Anne Love Forrest, of Washington, D. C. (a descendant of Uriah Forrest, of Revolutionary fame), was born in Richmond, Va., on May 3d, 1850. He was left an orphan at the age of four, and received his early education at a private school kept by his uncle, Mr. W. B. Johnson, of Virginia. Later, his uncle, Mr. Charles W. Forrest, of Washington, D. C., sent him to the Preparatory and Collegiate Departments of Columbian College (now George Washington University), from which he was graduated in 1871 with high honors, receiving the degree of Ph.B. While at college he developed a taste for engineering, and won all the prizes offered in that Department.

The principal engineering works in which Mr. Johnson was engaged were as follows:

Assistant Engineer on the extension of the Chesapeake and Ohio Railroad; in full charge of operations for the Chestnut Hill Iron Ore Company, of Columbia, Pa., at mines in Carroll County, Maryland; Superintendent of the Bachman Valley Railroad; Assistant Engineer and, afterward, Resident Engineer on the construction of the New York, West Shore and Buffalo Railroad; Resident, Division, and finally Deputy (New York) State Engineer; and for four years, City Engineer of Utica, N. Y. He did important work in reporting on the Utica water supply; the Utica sewerage system; straightening the Mohawk River through Utica; constructing the filtration plant at Albany, N. Y., and constructing and designing other filtration plants; developing a new system of water supply for Troy, N. Y.; developing water power at Trenton Falls, N. Y.; constructing the overhead crossing of Genesee Street, Utica, N. Y., over eighteen railroad tracks; and making an exhaustive examination of the construction of the Rutland Canadian Railroad in Vermont, in defense of a lawsuit.

From 1894 until a short time before his death, Mr. Johnson made his home in Utica, N. Y. He was a public spirited man, and from the first had much to do with, and was largely instrumental in, bringing about many needed reforms in that city. He was broad-minded, well-informed, and had excellent judgment, and his assistance and advice were sought in the several municipal departments.

Mr. Johnson was an engineer of marked ability, and a conscientious public official. All who knew him were impressed with his untiring efforts to obtain the best results, regardless of how much such measures

* Memoir prepared by Daniel L. Mott, Esq., Utica, N. Y.

might affect his personal comfort, liberty, or career. He listened courteously to the opinions of others, and unhesitatingly turned them to good account when he could. He possessed a keen sense of justice, and was eminently fair in his treatment of contracts, whether written by himself or not; apparently bearing in mind that, as an engineer, it was his duty to consider the terms of the agreement, as written, as being equally binding on his client and the contractor. He was not guilty of burdening the contractor with engineering errors. He was considerate and helpful to those in his employ, and won their loyalty and respect. He was a good mathematician, and had excellent command of the English language.

In 1870, Mr. Johnson was married to Miss Mary M. Shriver, daughter of General Edward Shriver, of Frederick, Md., who survives him. He is also survived by one daughter, Miss Elizabeth Forrest Johnson, who is head of the Baldwin School, Bryn Mawr, Pa., in which school Mr. Johnson was greatly interested.

Mr. Johnson was elected a Member of the American Society of Civil Engineers on October 7th, 1903.

EMIL KUICHLING, M. Am. Soc. C. E.*

DIED NOVEMBER 9TH, 1914.

Emil Kuichling was born on January 20th, 1848, at Kehl, Germany. Owing to the failure of the Revolution of that year, his parents, Dr. Louis and Marie von Seeger Kuichling, moved in 1849 to Rochester, N. Y., where their son grew to manhood and spent the greater part of his life.

Mr. Kuichling received his early education in private schools. His taste for engineering showed itself when he was quite young. At the age of 14, he took the position of Chainman in the office of the City Surveyor of Rochester and in the following year became Rodman on the Western Division of the New York State Canals.

In order to complete his education, young Kuichling entered the University of Rochester in 1864, and, after taking the classical course, was graduated in 1868 with the degree of A. B. He stayed another year at this University, for a post-graduate course in engineering, on the completion of which he received the degree of C. E. While at the University his vacations were devoted to practical work in the Engineer Corps of the Western Division of the New York State Canals, and, after his graduation, he was appointed Assistant Engineer on this work.

Not satisfied with his theoretical education, Mr. Kuichling studied engineering, in 1870-73, at the Polytechnicum of Karlsruhe, Germany, at that time the most famous school of its kind in Europe, where he had Sternberg and Baumeister as instructors. After completing this course, he returned to the United States and, in 1873, was appointed Assistant Engineer on the Rochester Water-Works. In 1876, he was made Principal Assistant Engineer on these works, a position which he held until 1885, when he was elected a member of the Executive Board which had charge of the public works of Rochester. In 1887, he resigned from this office, in order to undertake the work of designing the sewerage system for the west side of Rochester. He was engaged in this work until 1890, when he was appointed Chief Engineer of the Rochester Water-Works, which position he held for ten years, until it was abolished by a new City Charter which went into effect on January 1st, 1900.

In 1900-01, Mr. Kuichling was employed on the New York State Barge Canal as Engineer of Water Supply. In 1901 he moved to New York City, where he opened an office as Consulting Engineer, and had at once a large practice. While still in the various positions he held in Rochester, he was consulted about many other water-works, sewerage systems, etc. During the period from 1882 to 1892, he acted

* Memoir prepared by Edward Wegmann, M. Am. Soc. C. E.

as Consulting Engineer to the New York State Board of Health. His advice was often sought in connection with hydraulic engineering works, and he was frequently employed as expert in important litigations.

Mr. Kuichling was married on January 28th, 1879, to Sarah Louise, daughter of John S. Caldwell, of Rochester, N. Y., who survives him.

He was a member of the American Water Works Association; the New England Water Works Association; the Public Health Association; the Rochester Engineering Society, of which he was the first President; the Rochester Academy of Science; the Delta Upsilon Fraternity and Phi Beta Kappa Society; all branches of the Masonic Fraternity; and the Engineers Club of New York.

Mr. Kuichling was one of the foremost hydraulic engineers in America. He was a great student and read and digested all the important articles, bearing on the specialty he had selected, which appeared in English, German, or French engineering literature. His memory was wonderful, and stored not only the interesting facts he had gathered, but the very places where they were mentioned. He was a born investigator, and probed to the bottom every subject he was called on to examine. An indefatigable worker, he rarely relaxed, and day after day, when engaged on some important piece of work, he would be at his desk or at his drafting-table until midnight. Above all, he was a man of absolute integrity and honor. In money matters, he was disinterested, almost to a fault. In dealing with subordinates, he had no patience with carelessness and negligence, but he had a kind heart and was always just.

In the death of Mr. Kuichling, the Profession has lost one of its brightest lights and a man of noble, sterling character. He always took an active part in discussing important engineering questions, and will be much missed at the meetings of the many technical societies to which he belonged.

Mr. Kuichling was elected a Member of the American Society of Civil Engineers on September 3d, 1884, and served as a Director in 1901-03, and as Vice-President in 1905-06.

HENRY FRANCIS LABELLE, M. Am. Soc. C. E.*

DIED DECEMBER 12TH, 1913.

Henry Francis Labelle was born in Montreal, Que., Canada, on January 31st, 1860. His father, Regis Labelle, was a lumber merchant, and was a native of Jirome, in the Province of Quebec. His mother, Lucille Marceau, was born at Lacolle, in the same Province.

After his graduation from St. Mary's College, Montreal, he entered the Montreal Polytechnic School, from which, in 1882, he received the degree of Civil Engineer.

Mr. Labelle's first employment was as Rodman and Draftsman for the Chicago, Burlington and Quincy Railroad. In August, 1883, he returned to Canada, and, after about a year in the Dominion Lands Office, entered the office of the Chief Architect, Department of Public Works, Canada, where he was employed until 1890, during the last two years as Assistant Superintendent of Public Buildings. From 1890 to 1895, he was employed as Assistant Engineer or Engineer in Charge of the design and construction of numerous municipal water supply systems in New York, Pennsylvania, and Maryland.

By disposition and training Mr. Labelle was peculiarly fitted for the varied and interesting problems of hydraulic and hydro-electric engineering design, and after once entering this branch of the Profession, he did not leave it.

He was for several years in the employ of the East Jersey Water Company, the latter part of the time as First Assistant Engineer in charge of the Montclair office and of construction work amounting to approximately $1 000 000. In 1895, and again in 1900, he was employed by the Susquehanna Power and Paper Company, on surveys and designs for a 35 000-h.p. development on the lower Susquehanna River, and in 1902 by other parties on investigations and studies of power possibilities on the same river at Turkey Hill.

In 1901, he was appointed Designing Engineer on a new water supply for Santiago, Cuba, for the Military Government. In the latter part of 1902, he returned to Cuba where, for some time, he was engaged by Havana interests as Engineer to report on or design water supply and electric power and light systems.

In 1904 Mr. Labelle accepted employment under the Philippine Government, first as Engineer in Charge of all irrigation work in the Islands, under the Director of Public Works, and later as Principal Assistant Engineer on the construction of the new Manila Water-Works. While in charge of the irrigation works, he prepared a system of irrigation laws for the Islands.

* Memoir prepared by John T. Whistler, M. Am. Soc. C. E.

In 1907, on account of threatened lung trouble, he was advised by his physicians to leave the Philippines. He returned to the United States, and located in Silver City, N. Mex., where he remained until 1911, when heart trouble compelled him to change from the high altitude to a lower one, and he moved to Albuquerque, N. Mex., where he remained until his death.

It was typical of his character that he wrote of his heart trouble in 1911, "I now have the choice of two ways to die. I can either remain in Silver City and die of heart trouble, or I can go to Albuquerque and die of lung trouble."

After several months of correspondence and inquiry as to his experience and qualifications, he was notified, on November 5th, 1911, of his employment by the Argentine Government to act as Adviser in the development of an Irrigation Bureau. It was just at this time that his complication of heart trouble had developed, and, much to his regret, he was compelled to decline the office.

In 1908, he was married to Miss Emily Rose Woodford, of London, England, who for several years had been a resident of Philadelphia, Pa., and who survives him.

Mr. Labelle was the author of several discussions published in the *Transactions*, and had also contributed to various current technical journals. Though of foreign birth, he took out naturalization papers and became a citizen of the United States in January, 1904.

Altogether, Mr. Labelle was one of the sweetest and most noble characters with whom it was one's privilege to associate. He never complained, rarely spoke ill of any one, and then only with that calm judgment which guided his whole career. He never was known to do any one the slightest injustice. Simply to have known him was a privilege, to have known him intimately was an inspiration.

Mr. Labelle was elected a Member of the American Society of Civil Engineers on April 6th, 1898.

FRANK PARSONS LANT, M. Am. Soc. C. E.*

Died June 30th, 1914.

Frank Parsons Lant was born at Nassau, N. Y., on May 24th, 1858, and was of Colonial Dutch descent.

His business career, which included many important positions, was begun on September 26th, 1881, when he was employed as Axeman, and, subsequently, as Rodman and Assistant Engineer on the location and construction of the West Shore Railway, holding the latter position until March 1st, 1884.

From June 1st, 1884, to December 31st, 1886, Mr. Lant was City Engineer and Surveyor of Hudson, N. Y., designing and constructing sewers, highways, etc. He was also engaged in the private practice of surveying and engineering during that time.

During 1887, and until March 1st, 1888, he was employed on the location of a branch line for the Erie Railway, in Pennsylvania, and also made the surveys and had charge of the construction of the foundations for the Myrtle Avenue Elevated Railroad, in Brooklyn, N. Y. During this period he made the preliminary and location surveys for the New York and Massachusetts Railroad, from Ancram Lead Mines, N. Y., to near Westfield, Mass.

From March to November, 1888, he served as Division Engineer on the preliminary and location work for a line over the Blue Ridge Mountains, north of Marion, N. C., for the Charleston, Cincinnati and Chicago Railway, and from November, 1888, to March, 1889, he was engaged on a revision of the location of the New York and Massachusetts Railroad from Ancram Lead Mines, N. Y., to Chicopee Falls, Mass. In April, 1889, he was placed in charge of the reballasting of 60 or 70 miles of roadbed for the Rome, Watertown and Ogdensburg Railroad.

In November, 1889, Mr. Lant went to Jamaica, West Indies, where he remained until July, 1891, making location surveys for the West India Improvement Company.

From August, 1891, to February, 1896, he was engaged in the private practice of engineering and surveying, during which time he made the plans for the disposal of ashes in the filling in around Riker's Island, a survey for a railway proposition on Long Island, etc., etc.

In connection with the public works of New York City, Mr. Lant served the City Government, as follows: From February, 1896, to October, 1897, he was Engineer Inspector in the Department of Public Works; from October, 1897, to July, 1898, Engineer of Grades

* Extract from memoir prepared by Gerald E. Hart, Esq., of New York City and Jacksonville, Fla., supplemented by information on file at the Society House.

(Commissioners' Engineer) on the electrification of the Second Avenue Railroad; from August, 1898, to February, 1902, Transitman and Computer in the Topographical Bureau of the Board of Public Improvements; and from February, 1902, to March, 1907, Transitman and Computer in the office of the Engineer of Street Openings.

In March, 1907, he was appointed Manager (which title also included that of Chief Engineer and Surveyor) of the Lawyers' Engineering and Surveying Company, of New York City, which position he held at the time of his sudden death.

Mr. Lant was married on November 11th, 1891, to Stella, daughter of the late Jacob L. Seixas, who, with his father and mother, survives him.

He was an ardent student in his chosen profession, to which he did honor. His capabilities were such that his services and advice were frequently sought in cases of special importance. He possessed in a high degree the power to command loyal service from his subordinates, as well as the confidence and respect of all who knew him. His genial manner and kindly nature will long be remembered by many who, in his death, experience a sense of personal loss.

Mr. Lant was elected a Junior of the American Society of Civil Engineers on October 3d, 1888, and a Member on February 1st, 1910.

ULYSSES STANISLAUS LUTZ, M. Am. Soc. C. E.*

Died December 8th, 1913.

Ulysses Stanislaus Lutz, the son of Ignatius and Rose Jacwuin Lutz, was born in Philadelphia, Pa., on May 25th, 1853. He received his earlier education at Roth's Military Academy, and was graduated in 1873 at the head of his class from the Philadelphia Polytechnic Institute, with the degree of Civil Engineer.

In 1873, he was engaged as Rodman on the South West Pennsylvania and the Lewisburg, Centre, and Spruce Creek Railroads. In 1874 he served as Leveler on the Tyrone and Clearfield and New River Railroads; in 1875 as Rodman on the Pennsylvania Railroad; in 1876, as Transitman on retracing the old Portage Railroad; from 1877 to 1879, as Assistant Supervisor, Maintenance of Way, on the Pittsburgh and Philadelphia Division, of the Pennsylvania Railroad; in 1880, as Supervisor, Maintenance of Way, on the Shenandoah Valley Railroad; from 1881 to 1885, as Assistant Engineer on Surveys for the South Penn Railroad, for two years, and for two years as Resident Engineer in charge of Subdivision B, Division 5, of the South Penn Railroad, in Somerset County; in 1886, as Principal Assistant Engineer on the location and construction of the Bloomsburg and Sullivan Railroad; in 1887, as Principal Assistant Engineer on the location of the Baltimore and Drum Point Railroad; in 1888 as Construction Engineer of the 25th Ward Gas Works, in Philadelphia, Pa.; in 1889, as Division Engineer on the location and construction of the Cumberland Valley Extension of the Louisville and Nashville Railroad.

For a time Mr. Lutz followed private practice in Virginia and North Carolina, and, in 1892, became Constructing Engineer for the Pennsylvania Iron Works. In 1893 and 1894, he was Chief Engineer on the location and construction of the South Jersey Railroad, and thereafter was engaged on the construction of electric railroads in and around Baltimore, Md., having been, in 1895 and 1896, Manager of Construction for the Baltimore and Catonsville Railroad.

In 1897, he entered the service of the City of New York in the Department of Public Works, being first engaged on the Park Avenue Change of Grade Improvement from 56th to 96th Streets in Manhattan.

Mr. Lutz left the City's service to go with his former Chief, the late William F. Shunk, as Chief Assistant Engineer on the location and construction of the Quito and Guayaquil Railroad, in Venezuela,

* Memoir prepared by Charles W. Staniford and S. L. F. Deyo, Members, Am. Soc. C. E., and John H. Frazee, Assoc. M. Am. Soc. C. E.

and again to go with the Iowa and Minnesota Railroad, as Assistant Engineer on Construction.

In 1905, he entered the Department of Finance of the City of New York, as an Assistant Engineer, and held that position until his death, which occurred on December 8th, 1913, after an illness of three weeks.

Mr. Lutz's practical knowledge acquired through a long and varied engineering experience was of great service to the City. He was an indefatigable and conscientious worker, and those with whom he was associated were impressed with his sense of loyalty to the City and to his friends.

Mr. Lutz was elected a Member of the American Society of Civil Engineers on March 5th, 1912. He was also a member of the Masonic Fraternity.

WALTER ASHFIELD McFARLAND, M. Am. Soc. C. E.*

Died March 17th, 1915.

Walter Ashfield McFarland was born on May 31st, 1864, in Brooklyn, N. Y. He was educated in the public schools of that city and at Lehigh University, from which he was graduated in 1888, with the degree of Mechanical Engineer.

From 1888 until 1895, Mr. McFarland was engaged in engineering work under the Corps of Engineers, U. S. A., the first two years as Assistant Engineer on river and harbor work in New York and Connecticut, and, after 1890, as Resident Engineer on the Muscle Shoals Canal, Tennessee River, Alabama.

In 1896, he was appointed Superintendent of the Water Department of the District of Columbia, and held this position until his death. Soon after his appointment as Superintendent, he proceeded to reorganize the Department and re-arrange the system of distribution. The system, developed and constructed in accordance with his plans—which provided for the separation of the supply into pressure zones—includes the Brightwood Reservoir, the District pumping station, and the main trunk lines for the distribution. He attained notable success in reducing waste, as a result of which the mean daily consumption of water was reduced, in 10 years, about 6 500 000 gal., notwithstanding an increase of 60 000 in population served.

Mr. McFarland took a personal interest in every employee of the Department, from the common laborer upward, and received most loyal co-operation from them all. He had earned the respect and esteem of officials in other Departments of the District Government, with whom he was associated as a member of many commissions to investigate and improve methods in vogue, and his death, at a time when it would appear that many years of usefulness were still before him, was felt by each as a personal bereavement, as well as a distinct loss to the service.

Mr. McFarland took an active part in engineering circles, and at the time of his death, was President of the Washington Society of Engineers. He was also a member of the Cosmos Club and of the Southern Lehigh Club.

Pneumonia following influenza was the cause of his death, which occurred on March 17th, 1915. He was unmarried and is survived by an only sister.

Mr. McFarland was elected a Member of the American Society of Civil Engineers, on May 3d, 1910.

* Memoir prepared by D. E. McComb, M. Am. Soc. C. E.

DAVID ERNEST MELLISS, M. Am. Soc. C. E.*

Died March 24th, 1913.

David Ernest Melliss was born in New York City, on March 11th, 1848. His professional education was received at the School of Mines of Columbia College, in that city, where he pursued the regular course in Mining Engineering for three years. Early in 1867, he matriculated at the University of Goettingen, in Hanover, Germany, and, after two years, which were chiefly devoted to chemistry, physics, geology, and mineralogy, he was graduated with the degree of Doctor of Philosophy in 1869. The following year, he attended lectures in the Natural Science Course at the University of Vienna. He was afterward constantly engaged in professional work, chiefly as a Consulting Civil and Mining Engineer, in the course of which he examined professionally and reported on mineral lands in the United States, Europe, Mexico, and Central America.

In 1873, Mr. Melliss was in charge of a topographical and geological survey of the Provinces of Guanacasta and Nicoya, for the Costa Rica Government. In 1881, he was appointed Consulting Engineer to the Pacific Gas and Electric Company, of San Francisco, Cal., and in its interest visited the principal gas-works of the United States, England, France, and Belgium, for the purpose of studying and reporting on the different processes.

The plans for the buildings and wharves of the Union Iron Works of San Francisco were designed by him and carried out under his supervision, as were also those for the Arctic Oil Works, which were built of concrete masonry throughout and completed in 1884.

The water-works of Mazatlan, Mexico, with its pumping plant, two double reservoirs, 22 miles of steel mains, and the entire city distributing system, were designed by Mr. Melliss, and constructed under his supervision from the inception of the enterprise to its completion in 1890. For several years he acted as Consulting Engineer to the Guadelupe de los Reyes Mining Company, the most important mining concern in Sinaloa, and he was also the Administrator of the San José de las Bocas Mines.

He was also engaged in considerable work in Venezuela, and, in recognition of certain professional services rendered that Government, was decorated with the Order of the Bust of Bolivar.

While in Mazatlan, in 1899, he complied with the request of the United States Minister to Guatemala, Mr. Mizner, to accompany him and act as Secretary of Legation during his visit to Nicaragua and Costa Rica.

* Memoir prepared by Bolton B. Melliss, Esq.

Mr. Melliss' next important engineering work was the plant of the Olympic Salt Water Company, of San Francisco, representing an investment of about $500 000, every detail of which was designed and erected under his supervision.

In 1895, he acted as Superintendent of Works of the New City Hall Commissioners of the City of San Francisco, and at the same time followed his profession as Consulting Engineer. In this capacity (as Engineer), he remained until about 1904, when the pressing need of greater attention to his own affairs induced him to withdraw from active life at a comparatively early age. In this manner, he spent the remaining years of his life.

In the middle of March, 1913, he was stricken with pneumonia, from which he never recovered, his death occurring at Mill Valley, Cal., on March 24th.

Mr. Melliss was elected a Member of the American Society of Civil Engineers on October 2d, 1895.

WILLIAM EDWIN MOORE, M. Am. Soc. C. E.*

Died January 24th, 1915.

William Edwin Moore was born in Pulaski, near Newcastle, Pa., on May 6th, 1863. He attended the public school at that place and afterward worked his way through the Academy at Newcastle and the Pennsylvania State College. He was graduated from the latter in 1889.

The first few years of his career were spent in general engineering work, and from 1896 to 1898, he was in charge of surveying parties on the Canadian Pacific Railroad. At about this time, he became interested in hydraulic work and specialized in that branch of engineering.

In 1898, Mr. Moore took charge of a large power and irrigation development for the Lewiston-Clarkston Company, at Clarkston, Wash. This was one of the early developments of its kind in the United States, and the many new engineering problems presented were met and handled by him in the most efficient manner.

The Lewiston-Clarkston work was not only difficult, but covered nearly the whole field of engineering. It consisted in the construction of an auxiliary steam turbine plant of 1 500 h.p.; two hydro-electric plants, the larger having a capacity of 2 500 h.p., under an effective head of 485 ft.; 14 miles of wood stave pipe, through one of the most precipitous and rugged canyons in the Northwest; a pumping plant; and 25 miles of smaller pipe; also the construction of two concrete dams, one for storage and one for a head-gate reservoir, together with an elaborate settling basin and forebay.

The construction of the large wooden pipe line, although not the largest in the country, presented as many problems as any other known line. This may be better understood when it is realized that 13 miles of the line crossed 56 deep gulches, which called for many bridges and inverted siphons, several of the latter requiring heavy steel pipe in place of the wood stave. This system of water-works not only furnished power, but irrigated several thousand acres of land covered by the same project.

In 1908, Mr. Moore opened offices as a Consulting Engineer, in Spokane, Wash., and, later, became a member of the firm of the Ingersoll and Bugbee Engineering Company, of that city, where he practiced until his death.

Mr. Moore was not only thorough in technique and practical in his design, but had good executive ability in controlling and holding the confidence of all his employees. Those in his employ have often remarked that, no matter how startling a problem was presented to

* Memoir prepared by M. L. Bugbee, Cons. Hydr. Engr., Spokane, Wash.

him, his cool, decisive, and quick reply would make one believe that he had always been familiar with the occurrence.

He was deeply interested in the Spokane Association of Members of the American Society of Civil Engineers, and up to the time of his death had done much to promote a Water Code for the State of Washington.

Mr. Moore was a most genial man and respected by all who knew him. He found his chief pleasure in his home. He was an exemplary character, the very soul of honor, and prosecuted his work in a most zealous manner and with untiring energy.

He was married, in 1898, to Miss Evelyn King, of Boston, Mass., who, with two sons, Allen King Moore and Donald Edwin Moore, survives him.

Mr. Moore was elected a Member of the American Society of Civil Engineers on February 7th, 1906.

HENRY LEWIS OESTREICH, M. Am. Soc. C. E.*

DIED AUGUST 13TH, 1914.

Henry Lewis Oestreich was born in New York City, on February 13th, 1870. He was of German parentage, his father coming from Eckelsheim-on-Rhine, and his mother from Heidelberg. He received his early education in the public schools of New York City, and learned so rapidly that he completed his preparatory studies before he was old enough to enter college. At this period of his life, he was uncertain for what calling to fit himself. His own idea was to become a physician, but his mother wished him to be a civil engineer, and lived long enough to see him receive the degree of B. S. C. E. from New York University, in 1888. He was the youngest graduate in his class.

Mr. Oestreich's first engineering experience was gained during his last college vacation, as Chainman and Topographer on railway construction in Virginia. After his graduation, he was employed for about a year at general city engineering work in the office of Mr. B. Hufnagel, in Mt. Vernon, N. Y. In July, 1889, he entered the office of the late Charles B. Brush, M. Am. Soc. C. E., who lectured at New York University and knew of Mr. Oestreich's abilities as a student. From this time until April, 1897, he was engaged as Assistant Engineer in charge of various engineering works in and near Jersey City, including the construction of the Weehawken Viaduct, Weehawken Reservoir No. 2, Hillside Road, and part of the Hudson County Boulevard, including bridges and sewers.

In April, 1897, Mr. Oestreich entered the service of New York City as Assistant Engineer in the Department of Highways, where he remained in responsible charge of highway construction until May, 1900. He then secured a transfer to the New York Rapid Transit subway work, at that time just beginning under William Barclay Parsons, M. Am. Soc. C. E. He was engaged on this work until his death, 14 years later. Here his capacities were recognized by promotion, with the result that, during most of this period, his position was that of Resident Engineer in charge of important and complicated subway contracts; he stood at the head of a large corps of assistant engineers and inspectors.

Mr. Oestreich's qualities of mind and character were greatly admired by those who knew him. He was an indefatigable worker, painstakingly attentive to details, but never losing perspective on that account. His ready grasp and fair, common-sense treatment of an

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* Memoir prepared by F. C. Noble, M. Am. Soc. C. E.

engineering situation or matter in controversy could always be depended on. His ideals of duty and truthfulness were uncommonly high, and that they governed his every action was a fact to be appreciated in proportion to one's acquaintance with him. Added to these qualities, his sincerity, even temper, modesty, consideration, and courtesy endeared him to his friends and were recognized by all whom he met. It is certain that his progress in the Engineering Profession would have been continuous and rapid if it had not been for his untimely end.

Mr. Oestreich was married on June 10th, 1914, to Miss Stella B. Eccles, of Trenton, N. J., and his death was especially sad, as it occurred so soon after his marriage.

Mr. Oestreich was elected a Junior of the American Society of Civil Engineers on October 4th, 1892; an Associate Member on December 6th, 1899; and a Member on April 6th, 1909.

ARTHUR LOUIS PHILLIPS, M. Am. Soc. C. E.*

DIED JULY 13TH, 1914.

Arthur Louis Phillips was born in DuPage County, Illinois, on April 8th, 1864. He was educated at St. Ignatius College, Chicago, from which he was graduated in 1884. He immediately entered the service of the Chicago, Burlington and Quincy Railway, as a Rodman on construction in Missouri and Minnesota, and remained with that Company for two years. From 1886 until 1889 he was employed by the Northern Pacific Railway as Leveler on surveys in Montana and Washington. He was then promoted to the position of Resident Engineer. In 1889, he was appointed an Assistant Engineer on maintenance of way on the Southern Pacific Railway, with headquarters at Houston, Tex. In 1890-91-92 he was in the employ of the Denver and Rio Grande Railway as Resident Engineer and Locating Engineer.

On the opening of the Cherokee Strip, in Oklahoma, in 1893, Mr. Phillips was one of the first engineers in the field, and located many miles of the railway now forming parts of the Frisco System, being engaged on this work until July, 1899, when he entered the service of the Mexican Central Railway as Locating Engineer. He remained in that position until November, 1902, when he returned to Oklahoma and until March, 1903, was engaged on the location and construction of the Fort Smith and Western Railway.

In 1903 Mr. Phillips returned to the employ of the St. Louis and San Francisco Railroad, remaining with that Company until 1908, during which time he made surveys for the proposed extensions from Memphis to New Orleans, and was in charge of construction of the Frisco terminals in the latter city. He attained the position of Assistant Chief Engineer of the Frisco Railroad during this time, and made two trips to Mexico for the purpose of reconnoitering a proposed extension of the St. Louis, Brownsville and Mexico line from Brownsville to Vera Cruz. He was also sent to South America by some New York interests to look over a proposed railway in Colombia, which, however, was not built.

From November, 1908, until March, 1912, Mr. Phillips was engaged in private practice as Consulting Engineer in New Orleans, La. He then accepted a position as Chief Engineer of The Cuba Railroad, with headquarters at Camaguay, Cuba. He remained with this Company until June, 1913, and then resigned to accept a position as Assistant Chief Engineer of the Tela Railroad Company, in Honduras. On the completion of the location surveys for that work, he returned

* Memoir prepared by F. G. Jonah, M. Am. Soc. C. E., and Clinton H. Fisk, Assoc. M. Am. Soc. C. E.

to New Orleans, and in December, 1913, reopened an office as Consulting Engineer. Shortly thereafter his health began to fail, and the last few months of his life were spent at various health resorts, but he declined rapidly, his death occurring in Denver, Colo., on July 13th, 1914. The body was taken back to New Orleans for interment.

On September 30th, 1905, Mr. Phillips was married, in Chicago, Ill., to Miss Jessie Fleming, who survives him.

He was a most courteous and affable gentleman, very considerate to, and greatly beloved by, his subordinates. He was a splendid Spanish scholar, and his experience in Latin-American countries had enabled him to speak the language fluently.

Mr. Phillips was elected a Member of the American Society of Civil Engineers on November 4th, 1903. He was a Thirty-second degree Mason and a Shriner, and was also a Member of the American Railway Engineering Association.

ROBERT LELAND READ, M. Am. Soc. C. E.*

DIED JUNE 9TH, 1912.

Robert Leland Read was born in Manchester, N. H., on July 12th, 1841. His ancestors were distinguished in the early history of the country and, in consequence, he became a member of the Society of Colonial Wars and several other prominent organizations interested in the Colonial and Revolutionary periods. After his retirement from the active duties of his Profession, he was frequently present at their meetings in Boston, and was always actively sympathetic in their proceedings.

Mr. Read was educated at Dartmouth College, and was one of the early graduates of the Chandler Scientific School. In 1864, there were seven graduates, Mr. Read being the only one of that number who entered the Profession of Civil Engineering.

Soon after leaving his studies, in January, 1865, he was fortunate in securing a position as Assistant Engineer on some surveys connected with the Indianapolis, Cincinnati and Lafayette Railroad, and he remained with this road for several years. The Civil War was just ending, and the railroads were passing through a trying period, with little money available either for maintenance or construction; it is rather surprising, therefore, to find that the compensation given Mr. Read was $100 per month and all expenses paid. A short time afterward, we find, from his notes, that the regular pay for an instrumentman was $130 per month and expenses. The panic of 1873, however, caused these prices to drop suddenly and permanently.

The early railroad experiences of Mr. Read follow closely the writer's experiences in Southern Indiana during the same period, and are so characteristic of local conditions of that time, that he has ventured to rehearse some of Mr. Read's notes and later to quote several lines in full.

A majority of the railroad officials prided themselves on being practical men, with no scientific training. They looked on an engineer as a necessary evil in construction, to be quickly discharged as soon as the roadbed was finished. The master mechanics were generally promoted from the ranks of locomotive engineers and, as a rule, had but a poor opinion of book learning. The master mechanic of Mr. Read's road once said to him: "I never bother myself with what other master mechanics are doing, except to make my engines different from theirs, so the president and superintendent of our road will think they have a smarter man than the other roads have."

Under this system, when the "Big Four" roads were consolidated, out of 500 locomotives, there were 115 different types. They were small wood burners, capable of drawing only from three to four cars.

* Memoir prepared by Desmond FitzGerald, Past-President, Am. Soc. C. E.

Soon after Mr. Read began his railroad work, he was sent to make surveys of a branch line from Fairland to Martinsville. In regard to this branch, he wrote:

" * * * the first twelve miles being new line and the last twenty-six the rebuilding of an old road that had been originally laid with strap rail and abandoned many years previously when the track became so bad that a man had always to walk ahead of every train with spikes and maul to fasten down the loose rails.

"The Resident Engineer ran the transit and I the level. Just before the location was finished, the Resident Engineer went on a spree and I never saw him on the work afterward. The Chief Engineer, who only spent a few hours on the work during the whole period of construction, placed me in charge, but would give me no assistant, so I had to do all the instrument work, inspection, and office work; this meant at least sixteen hours a day. Our party was not allowed a horse, so we had to walk the thirty-eight miles between the ends of the work, living on the country, with all that that meant in Southern Indiana, which was principally settled by poor whites from Kentucky."

Following these early experiences, Mr. Read became Chief Engineer of several railroads connected with the "Big Four" system. From October, 1870, to May, 1871, he was engaged as Contractor on the construction of twenty-nine bridges on the Atchison, Topeka and Sante Fé, after which he was connected with the Ohio and Mississippi Railroad, and the construction of the Storrs Incline at Cincinnati. Later, he became Chief Engineer of the Indianapolis Belt Railway, and, in the latter capacity, was associated with General Harrison, the Attorney for the enterprise, who, later, became President of the United States.

On September 2d, 1869, Mr. Read was married to Miss Abby H. Eastman, of York, Me., the daughter of Dr. Caleb and Adeline Talpry Eastman. She died on August 30th, 1893.

When Mr. Read retired, he removed to Malden, Mass. He was a prominent summer resident of Biddeford Pool, Me., where he was active in good works of every kind. In his will there was a bequest of $1 000 to Dartmouth College, "the income of which is to be used as an annual prize for the best work in descriptive geometry."

The foregoing are the hard and fast outlines of a busy life, but they poorly portray the real man, whose kindness and charm of manner endeared him to all with whom he came in contact. The latter part of his life, after the struggle and heat of the day had passed, must have been in many ways very lonely, but he was never heard to complain, and maintained a calm and cheerful spirit to the end. He died in Portland, Me., on June 9th, 1912.

Mr. Read was elected a Member of the American Society of Civil Engineers on September 2d, 1874, and served as a Director in 1892 and 1893. He was associated with other Engineering Societies, and served a term as President of the Engineers' Club of Cincinnati.

HENRY MARTYN ROOD, M. Am. Soc. C. E.*

DIED DECEMBER 4TH, 1914.

Henry Martyn Rood, through a long line of Colonial ancestors, was descended from the illustrious company of pioneers who, imbued with high religious ideals, and seeking an enlarged freedom of life and conscience, braved the perils of a storm-tossed ocean, an inhospitable wilderness, and a barbarous people, came in the *Mayflower* and, settling in New England, stamped largely on the New World their most virile characteristics of love and zeal for righteousness.

Many years afterward Mr. Rood's father, the Rev. David Rood, inspired with the fervid religious zeal of his ancestors, determined to carry the Gospel of Jesus Christ to the savage tribes of the dark continent of Africa, and so Henry Martyn Rood was born in Amanizimtoti, Natal, South Africa, on February 21st, 1853.

Some of the reminiscences of Mr. Rood's early life were most interesting. From necessity, the companions of his early years were the native Zulu boys of whom he spoke as being highly intelligent, indeed they considered themselves as quite superior to boys of the white race, who required many years of schooling, while they "to the manor born" knew it all instinctively, and felt all the contempt of a dominant race for a weaker and inferior one, and ofttimes jibed the lonely small boy by calling him "white", which was as great a stigma as "nigger" applied to the blacks.

The Rev. Mr. Rood, a graduate of Williams College, and a scholarly man, desiring for his son a liberal education, sent him in 1871 to Phillips Academy, Andover, Mass., from which, in 1873, he entered Yale where he took the regular classical course and was graduated with honors in the Class of 1877.

During his college course, in which he applied himself assiduously to his studies, he became particularly interested in mathematics, and attaining great proficiency in that department, was awarded prizes for superior excellence.

In these college years, his affable disposition and studious habits won for him many staunch and enduring friends. At the conclusion of his college course, he, like many another scholar, soon found he lacked the practical knowledge that would enable him to make a livelihood. Being in ill-health, he returned to the land of his birth and after a time obtained employment with a Government Surveyor.

Probably Mr. Rood's brief experience at this period as Assistant Land Surveyor (appealing to his fondness for mathematics, and affording healthful occupation for which he had great need), determined his

* Memoir prepared by F. S. Odell, M. Am. Soc. C. E.

future career, for shortly afterward he returned to the United States and, entering the Rensselaer Polytechnic Institute, completed the engineering course in two years, receiving his degree in 1883. He soon took a position on the Burlington and Missouri Railroad, in Nebraska, and did pioneer work as Transitman and Leveler on location and construction for about one year.

This class of work, with the attendant hardship and exposure, proved too strenuous for the youth, whose health and physical vitality was considerably below the normal. He therefore resigned this position and on his return to the East was for some years associated with the late J. James R. Croes, Past-President, Am. Soc. C. E., as Assistant on surveys for the New Rochelle Water Company's earliest works and on the construction of the Suburban Rapid Transit Company's railway which was then penetrating the "annexed district", now the Borough of The Bronx.

This work included the building of the bridge crossing the Harlem River, at the head of Second Avenue, and extending the line over a right of way purchased and owned by the Company through solidly built-up city blocks. This portion of the road was supported on solid brick piers elevated sufficiently to cross intersecting streets above grade. It was designed for heavy rolling stock, and was operated by steam locomotives far more powerful than any hitherto used on other elevated lines in the city. Incidentally, it may be remarked that this stronger and heavier equipment proved its superiority at the time of the great blizzard of March 13th, 1888, when every other line of transportation, both surface and elevated, in New York City, was blocked with trains everywhere stalled and unable to proceed because of the accumulated masses of driven snow.

After leaving the Suburban Rapid Transit Railway, Mr. Rood was connected for a time with the construction of the second Croton Aqueduct as Engineer for Brown, Howard and Company, Contractors, on a section of the tunnel work. His next work was on the New York State Canals, as Assistant to Ellis B. Noyes, M. Am. Soc. C. E., Division Engineer, who was in charge of improvements on the Champlain Canal, lengthening locks, building bridges, and in various ways increasing the capacity and rendering the waterway more easily navigable.

Mr. Noyes writes, "Mr. Rood, as I recollect him, was of a very retiring disposition, and made few friends outside of those with whom he was brought in contact by his work. Those who did get to know him, liked him very much, and his professional work was always very carefully performed, he being painstaking and careful."

In August, 1891, the appropriation for the work was exhausted, and the engineering party disbanded. Shortly afterward, Mr. Rood became associated with Mr. Purdy, an Architect and Engineer, in

Chicago, Ill., where he was engaged in the computations and details of designs of high buildings.

During these migratory years, so well known to perhaps the majority of young engineers, but little time elapsed between successive engagements, due very largely to the ability and fidelity with which he invariably performed his part in each.

About this time (1894) Mr. Rood was married to Miss Grace Sarah Mellen, and located at Mount Vernon, N. Y., where he became Acting City Engineer in the Department of Public Works. Mount Vernon had recently become a city, and, in the many improvements inaugurated then and executed in the ensuing years, the industry, integrity, and ability of Mr. Rood were put to the test and never found wanting.

A political revolution in 1897 resulted in deposing the Commissioner of Public Works, and Mr. Rood lost his position as City Engineer.

In the succeeding years, his services were sought for various municipal works, including water-works for Kingston and White Plains, and paving and sewerage at Port Chester, N. Y., where he had been associated for some time with the writer.

The works of the Municipal Engineer, having to do so largely with sub-surface structures, such as aqueduct tunnels, subways and conduits for electric cables, pipes for sewers, water, gas, and steam, foundations for pavements, piers and abutments, although inconspicuous and unobtrusive, yet minister in the highest degree to the health and well-being, as well as to the convenience and comfort, of modern life, so Mr. Rood, being of an exceedingly modest and unassuming disposition, shunning rather than courting the limelight of popular acclamation, patiently and quietly wrought into the fabric of his life those homely, sterling characteristics and high ideals that make life healthful, helpful, and hopeful, ever radiating an influence for righteousness. He is survived by a widow and five children.

Mr. Rood was elected a Member of the American Society of Civil Engineers on December 3d, 1890.

GEORGE FREDERIC SIMPSON, M. Am. Soc. C. E.*

DIED APRIL 23D, 1915.

George Frederic Simpson was born at Poughkeepsie, N. Y., on February 13th, 1854. His father died while the boy was quite young, and he and his brother were left to the care of their mother. In 1874, Mr. Simpson won a State Scholarship to Cornell University, and, in 1879, he was graduated from the Department of Civil Engineering, having supported himself during his entire college course.

During the summer of 1880, he was employed as Levelman and Draftsman on the Pueblo and Silver Cliff Railway. In the fall of that year, he was appointed Chainman and Acting Rodman on the New York, Ontario, and Western Railway, remaining in this position until March, 1881.

From March, 1881, to June, 1883, Mr. Simpson was engaged, for the New York Steam Company, on surveys, maps, laying steam mains, inspecting iron and stone for steam boiler stations, etc.

In June, 1883, he was appointed Assistant and Resident Engineer on the construction of the foundation and pedestal of the Statue of Liberty, on Bedloe's Island, New York Harbor. In this capacity, Mr. Simpson inspected the materials and workmanship, and designed and made the working drawings of the final design for the pedestal. The block of concrete used in the base of the Statue was the largest single block ever made at that time. He remained on this work until May, 1886, when the foundations and pedestal were completed.

Mr. Simpson then entered the employ of the Otis Elevator Company. His most important work in this position was his design of the curved portion of the elevator shaft of the Eiffel Tower, which had puzzled the French engineers. He also designed the elevators for the tower of the World Building in New York City.

In 1888, Mr. Simpson entered the service of the New York Elevated Railroad Company, and was placed in charge of the First Section of that road. In 1891, he was appointed Chief Engineer of the Harriman Coal and Iron Railroad, with headquarters at Harriman, Tenn., which position he retained until January, 1893, when he went to Niagara Falls, N. Y., to take charge, for the Cataract Construction Company, of the design of the stone tunnel intersections and building details. He also superintended the erection of the filtration plant and pumping station for the Contractor of the Water Company at the same place, and, as Assistant Engineer in the United States Force-at-Large, he superintended the fleet of drill boats, dredges,

* Memoir prepared by the Secretary from information on file at the Society House.

scows, etc., used in submarine blasting and dredging at Strawberry Island.

In 1899, Mr. Simpson returned to New York City. For a number of years he had been interested in the development of subways, which he held to be the only solution for handling the growing traffic of the city, and, in February, 1901, when the construction of the first subway was begun, he was appointed an Assistant Engineer on the staff of the Chief Engineer of the Board of Rapid Transit Railroad Commissioners, under which this work was to be done. Mr. Simpson proved to be one of the ablest engineers engaged in the work, and, in June, 1905, he was detailed, with others, to investigate and report on the question of the ventilation of the subway and on traffic observations on city transportation lines.

When the Board of Rapid Transit Railroad Commissioners was abolished by the Act of July 1st, 1907, and the Public Service Commission established, Mr. Simpson was appointed an Assistant Engineer in the First District of the Commission. In 1908, he was transferred to the Sixth Subdivision of Design. This Department has supervision of investigations of, and designs for, ventilation and drainage, structural plans for station changes, and electrical conduit work, done under contract for the Commission. In 1910, he was placed in charge of this Subdivision and, on August 1st, 1914, was appointed Designing Engineer, which position he held at the time of his death, which occurred on April 23d, 1915, from a sudden relapse after an attack of the grippe from which he had apparently recovered. He is survived by his mother, two sons, and three daughters.

Mr. Simpson's engineering ability was well known and recognized among the Profession and especially by his associates in the Public Service Commission. He was a man of kindly, simple, and true character, absolutely incorruptible. Working always with unselfish devotion and unwavering fidelity, without regard for reward, and for the best interests of the State and City, his life should be an inspiration and example to those engaged in public service.

He was a Charter Member of the Cornell Society of Civil Engineers and served as its Vice-President in 1907-08.

Mr. Simpson was elected a Member of the American Society of Civil Engineers on March 2d, 1887.

CLINTON FITCH STEPHENS, M. Am. Soc. C. E.*

DIED MAY 12TH, 1915.

Clinton Fitch Stephens, the son of Nelson Timothy and Elizabeth Rathbone Stephens, was born in Cayuga County, New York, on October 27th, 1847. His native gifts marked him as an engineer, for from his earliest years he had unusual mathematical and constructive abilities. Ready for technical training when he was thirteen or fourteen years old, he was obliged to delay beginning such studies, for the Rensselaer Polytechnic Institute, into which he sought admission, published as one of its entrance conditions that a student must be at least sixteen. The years he waited for ripening, he devoted to reading law in his father's office at Auburn, N. Y., and in helping care for his father's farm near Canandaigua. He entered Rensselaer in 1864.

In the summer of 1867, Mr. Stephens was with the late Theodore G. Ellis, M. Am. Soc. C. E., on the Connecticut River improvement surveys. From Rensselaer, where he had taken the Mining Engineering Course, he, in 1868, turned westward, and from December, 1868, to December, 1869, was Assistant Engineer on the Des Moines Railway, in Iowa. From December, 1869, until January, 1872, he served as Locating and Division Engineer on the Louisiana and Missouri River Railway, in Missouri. During 1872, he was Division Engineer on the Cairo and St. Louis Railway, in Illinois, and, during 1873 and 1874, Draftsman and Division Engineer on the St. Louis, Iron Mountain and Southern Railway, in Missouri, in charge of the construction of the Cairo Division. From April to December, 1875, he was Chief Engineer of the St. Louis and Eastern Railway, in Illinois, and then financial troubles stopped the work. From December, 1875, until June, 1876, he was Division Engineer on the Little Rock and Fort Smith Railway, in Arkansas, and in June, 1876, he became Chief Engineer of the Dallas and Wichita Railway, in Texas.

In 1877, Mr. Stephens was chosen Chief Engineer of the Texas and St. Louis Railway, now better known as the "Cotton Belt Route" and the St. Louis Southwestern Railway. He constructed all the original road from Birds Point, in Missouri, to Gatesville, Tex. In this work he bridged the Arkansas River at Pine Bluffs; and also the Red River at Texarkana, where it had been claimed a bridge could not be built. In those days, when methods of constructing cement foundations below many feet of surging sand and water, and working nights with electric lights, were new and wonderful, this latter work had considerable press comment and illustration, and some pageantry

* Memoir prepared by Miss Kate Stephens, New York City.

at its completion. Stephens, a town in Arkansas, was in those days named after him.

After building and managing the "Cotton Belt" for a time, Mr. Stephens resigned his duties in 1885, and gave his time to the Southwestern Lumber and Timber Association, of which he was President. After an interim of some years, however, he again took charge of construction—this time of the St. Louis, Belleville and Southern Railway, which was absorbed later by the Illinois Central. In 1898, he became Manager of the Mine La Motte properties—mills, smelters, commissary stores, 4 mines, and 125 farms. Under his direction the earning power of Mine La Motte was increased so greatly that after five years the property sold at a notable advance on its purchasing price. In 1907 and 1908, Mr. Stephens was in the Philippines and China, as Consulting Engineer, and also in charge of certain constructions. To sum up his professional labors: He located and constructed 2 000 miles of railway in North America and the Philippines, operated certain roads, and successfully managed large mining and smelting properties.

Like some other Americans of the oldest and best Anglo-American blood, Mr. Stephens was "objective" in business matters, that is, he had that quality which has been called a characteristic of educated and cultivated English stock—the quality of looking out for the other party in business transactions. This is not the method most followed in the United States in these days, and Mr. Stephens made many a contract in which he was the loser because of smaller, craftier men seeking only their personal advantage; but he had at least the satisfaction of constancy to the ideals of his forbears and race. He was large-souled, generous, and helpful, and many a young engineer seeking a start found it through his effort to place him.

He was twice married, and had the gratification, before he died, of knowing that his grandson had begun his studies as an engineer.

Mr. Stephens was elected a Member of the American Society of Civil Engineers on September 5th, 1877.

LUCIAN ARNOLD TAYLOR, M. Am. Soc. C. E.*

DIED NOVEMBER 19TH, 1914.

Lucian Arnold Taylor, the youngest son of Jared and Catherine (Truesdell) Taylor, was born at Harrisville, R. I., on June 20th, 1846. Until he was sixteen years of age, his education was confined to winter schooling, the remainder of the year being spent on his father's farm at Woodstock, Conn.

At the outbreak of the Civil War, Mr. Taylor tried to enlist but, being only fifteen years old, he was not allowed to do so until July 15th, 1862, when he enlisted in Company B, 18th Connecticut Volunteers, and served until the close of the war, in 1865. He was in active service all the time except for about four months, when he was confined in the Southern prison at Richmond and at home recovering from that experience. At the close of the war, he returned to Woodstock and spent the next two years working on the farm and attending Woodstock Academy.

In 1867, Mr. Taylor moved to Worcester, Mass., and attended Howe's Business College. In 1868, he entered the employ of the City of Worcester, as an Assistant to the City Engineer, Mr. Phinehas Ball. He remained in the City Engineer's office until 1882, having direct charge of several important pieces of work, chief among which were the rebuilding of the Leicester Dam after its failure in 1876; the construction of the Island sewer, involving heavy and difficult work, in 1878 and 1879; and the building of the first Holden Dam, in 1880.

In 1882, Mr. Taylor was elected Water Commissioner, continuing in that office until 1885 when he resigned to enter the contracting firm of William C. McClallan and Company, of Boston.

After his partner's death, in 1886, Mr. Taylor bought Mrs. McClallan's interest in the firm, and from that time until his death carried on the business as a Consulting Engineer and Contractor. The water and sewerage systems of many New England cities and towns were built by him, among the most important being those of Bar Harbor, Kingfield, Wells, and Cumberland, Me.; Newport, N. H., St. Albans and St. Johnsbury, Vt.; Lynn, Orange, Quincy, Webster, Millbury, Concord, Provincetown, and Rutland, Mass.; and New Haven and Groton, Conn.

The fact that many of Mr. Taylor's contracts included both the engineering and the construction indicates the high esteem which his clients had for his judgment and integrity. Often he was invited to bid without competition or with only a limited number of bidders,

* Memoir prepared by C. M. Allen, M. Am. Soc. C. E.

and so satisfactory were his services that, in later years, many of his engagements were in communities where he had formerly been employed.

During the later years of his life, he did less contracting and more consulting work. When forced by illness to give up his practice in 1913, he was Consulting Engineer for the City of Lynn, and the Town of Falmouth, Mass., and had been President of the Millbury, Mass., Water Company for several years.

Although Mr. Taylor's office was in Boston for the last twenty-nine years of his life, he retained his residence in Worcester, where his death occurred on November 19th, 1914, after an illness of more than a year, having been caused by a complication of diseases.

He was married on August 29th, 1868, to Jeannette Arnold, of Putnam, Conn., who, with one son and one daughter, survives him.

He was a member of the Boston Society of Civil Engineers, the Engineers Club of Boston, the American Water-Works Association, the New England Water-Works Association, the Worcester County Society of Engineers, the Worcester County Mechanics Association. and the George H. Ward Post, G. A. R.

Mr. Taylor was elected a Member of the American Society of Civil Engineers on November 4th, 1891.

LAWRENCE CALVIN BRINK, Assoc. M. Am. Soc. C. E.*

Died May 2d, 1914.

Lawrence Calvin Brink was born at Frenchtown, Hunterdon County, N. J., on September 16th, 1869. His parents were Stacey Bray and Henrietta Waterhouse Brink. His father was descended from the early Dutch settlers of New York and New Jersey. His mother's ancestors came from England and settled in New Jersey during the early part of the Eighteenth Century. A great-grandfather, on his mother's side, Joshua B. Calvin, was a descendant of John Calvin, the reformer. Joshua B. Calvin was at one time a member of the Pennsylvania Legislature. Another ancestor on the maternal side was Dr. Benjamin Waterhouse, who was born in Newport, R. I., and is said to have been the physician who introduced vaccination in America. He was a close personal friend of George Washington, and a prominent surgeon in the Continental Army. He taught in Harvard College from 1783 to 1812 as Hersey Professor in Theory and Practice of Physics. Mr. Brink was a nephew of John Waterhouse, M. Am. Soc. C. E., a prominent engineer, who for many years was Chief Engineer of the Manhattan Elevated Railway system in New York City.

After being graduated from High School, at Trenton, N. J., Mr. Brink, in September, 1890, entered Lehigh University, from which he was graduated in 1894 with the degree of Civil Engineer. Before entering college, he had served for a short time in one of the departments of the Pennsylvania Railroad Company.

Soon after his graduation, he became connected with the Engineering Department of the New York Central and Hudson River Railroad Company under Walter Katté, M. Am. Soc. C. E., Chief Engineer, remaining there until he resigned in January, 1900. He was assigned first as Rodman in charge of a line and grade party, on the construction of the new four-track bridge over the Harlem River, at Park Avenue, New York City, and the elevation of the tracks north of it. He entered into this work with the earnestness and enthusiasm which was so characteristic of him, and proved himself to be a young man of unusual ability. His relations with those with whom he was associated there were of the most pleasant character. From October, 1895, to December, 1897, as Junior Assistant Engineer, he was engaged, under the direction of Ezra B. Naylor, M. Am. Soc. C. E., an Assistant Engineer of the Company, on the Park Avenue improvement and on surveys and studies for future improvements. From then until January, 1899, Mr. Brink was engaged on general work on the lines east of

* Memoir prepared by Jacob Stinman Langthorn and Robert Ridgway, Members, Am. Soc. C. E.

Buffalo. Until January, 1900, he was the Assistant Engineer in direct charge of construction, under the Maintenance Department, on the main line between New York City and Albany, and on the Harlem and Putnam Divisions, making surveys, masonry designs for the renewal of bridges, and studies for changes of alignment. During this time, he had direct charge also of dock and yard property in New York City, including the dredging of slips and the extension and reconstruction of piers.

After leaving the New York Central and Hudson River Railroad Company, he was in the employ of Mr. Henry Mesa, Civil Engineer, of New York City, on general surveying and engineering work, until January, 1901.

On January 8th, 1901, Mr. Brink entered the service of the Rapid Transit Railroad Commission, as an Assistant Engineer on the construction of the Rapid Transit Subway in New York City, and was assigned to Section 5-A, which included the work from 41st Street and Park Avenue, through 42d Street, and north on Broadway to 47th Street. For the first two years, he was in charge of laying out this intricate and difficult work, and from that time, until its completion in the latter part of 1904, was in direct charge of the construction of the section. The value of the work under his charge was about $3 500 000, and his handling of the job won the warm commendation of his superiors. On the completion of Section 5-A, he was assigned to other work, and made studies for the ventilation of the subways. In connection with this, he designed the louvres which are a feature of the ventilating system in use to-day.

On January 24th, 1906, he left the service of the Rapid Transit Commission to accept the position of Night Superintendent with the United Engineering and Contracting Company, and in that capacity was engaged on the Company's contract for the construction of the Pennsylvania Tunnels under 32d and 33d Streets, New York City, between Seventh Avenue and the East River. He left the Company's employ in April of the same year.

In the following month, May, 1906, Mr. Brink was appointed Assistant Engineer by the Board of Water Supply, City of New York, and assigned to the Wallkill Division of the Northern Aqueduct Department, with headquarters at New Paltz, Ulster County. He was in direct charge of the studies for the location of the Aqueduct, and in April, 1907, was promoted to the grade of Division Engineer in charge of the Division. The Wallkill Division, as finally located, comprised about 12 miles of the Catskill Aqueduct, which was designed for a daily capacity of 500 000 000 gal. The work included a portion of the grade tunnel through the Shawangunk Mountains, 6¾ miles of cut-and-cover aqueduct on the slope of those mountains and on the east side of the Wallkill Valley, and the deep pressure tunnel under the valley

of the Wallkill River. This pressure tunnel, which is 4.4 miles long, was the most important feature of the work. It was constructed from six shafts, ranging in depth from 340 to 480 ft., three of them being circular in section, lined with concrete, and to be maintained as permanent shafts. The tunnel reached a maximum depth below the hydraulic gradient of about 520 ft. The cost of constructing the work on the Wallkill Division was approximately $5 500 000, and it was finished well within the preliminary estimate.

Mr. Brink remained in charge until the work was well advanced, but not until it was completed, resigning in March, 1910, to become General Manager and General Superintendent of the Pittsburg Contracting Company, and continued in that position until his death. Under his direction, the Company completed Contract 52 of the Catskill Aqueduct, in the White Plains Division of the Southern Aqueduct Department. This contract cost about $2 000 000, and comprised about 2.8 miles of cut-and-cover construction, and two difficult grade tunnels in earth, one of them requiring compressed air for its prosecution. He also directed for the Company the work on Contract 65 of the Catskill Aqueduct, one of the four contracts comprising the deep pressure tunnel, 18 miles long, of the City Aqueduct Department. Contract 65 extended from Aqueduct and Burnside Avenues, in the Borough of The Bronx, to Central Park, near Eighth Avenue and West 99th Street, Borough of Manhattan, including Shafts 6 to 12, inclusive, and about 5½ miles of tunnel. Its estimated cost was more than $5 000 000. Mr. Brink organized the force for this work, and supervised the construction of it until his last illness. At the time of his death, the excavation was nearly completed, and the concrete lining was well advanced. As General Manager of the Company, he enjoyed the distinction of holing through the longest stretch—18 miles—of continuous tunnel in the world. He also managed for the Company a tunnel contract, for the Passaic Valley Sewerage Commission, near Newark, N. J., and assisted in preparing bids for a number of large construction contracts.

In the spring of 1914, he contracted typhoid fever, and, after an illness of five weeks, died on May 2d, 1914.

His long experience on large engineering works in the vicinity of New York City, added to his previous training and his sterling qualities as a man, fitted him for leadership in his Profession, and made him well known and respected among engineers and contractors in and about the City. Mr. Brink was an original thinker, possessing a true scientific mind; he was an earnest and faithful worker, an interesting companion, and a loyal friend. By his human, modest, and lovable qualities, he endeared himself to a large circle of friends, among

whom were many of his employees, in whose welfare he always took a warm interest.

He was a Member of the Municipal Engineers of The City of New York and of the Beta Chi Chapter of Phi Gamma Delta Fraternity. He was married, on January 22d, 1896, to Miss Belle Myra Young, of Baltimore County, Md., who survives him.

Mr. Brink was elected an Associate Member of the American Society of Civil Engineers on October 7th, 1908.

JAMES WILHELM CARPENTER, Assoc. M. Am. Soc. C. E.*

DIED JUNE 10TH, 1915.

James Wilhelm Carpenter, a grandson and namesake of James Henry Wilhelm, one of the pioneer officials of the Lehigh Valley Railroad, was born at Mauch Chunk, Pa., on June 16th, 1885.

In December, 1901, Mr. Carpenter was employed by the Chief Engineer of the Central Railroad Company of New Jersey on the restoration of the Lehigh and Susquehanna Division after the disastrous freshet which occurred at that time. He remained with the Company until 1906, having been engaged on bridgework, pile-driving, surveying, dredging, construction of freight and coal dock, freight yard, etc.

In September, 1906, he went to Sonora, Mexico, with the Cieneguita Copper Company of New York City. He made surveys and a map of the Company's property and supervised the construction of its various buildings, etc., returning to the United States in the summer of 1907.

In August, 1907, Mr. Carpenter was appointed Engineer in entire charge of all field work for the Cleveland Electric Illuminating Company during the construction and equipment of its Lake Front power-house and several other structures. During the past year, he had had charge of the valuation of the entire equipment of the Company and had almost completed this work at the time of his death which occurred at his home in Cleveland, Ohio.

On October 9th, 1912, he was married to Miss Gertrude Easton Blakslee, a granddaughter of James I. Blakslee, at one time Superintendent of the Lehigh Valley Railroad and an associate of the late Asa Packer in its construction. Mr. Carpenter is survived by his widow and two children, Jean Alice Carpenter and James Wilhelm Carpenter, Jr.

He was a young man of unusual ability and good principles, quiet in manner, and possessed of an exceptional personality.

Mr. Carpenter was elected a Junior of the American Society of Civil Engineers on February 28th, 1911, and an Associate Member on June 24th, 1914.

* Memoir prepared by I. M. Church, Esq., Mauch Chunk, Pa.

HORACE ARTHUR COOK, Assoc. M. Am. Soc. C. E.*

Died May 17th, 1914.

Horace Arthur Cook was born in Miller, Ind., on April 4th, 1877. He was graduated from Valparaiso University, in 1896, with the degree of B. S., and in the Fall of that year began work as Rodman on road construction in Lake County, Indiana. From September, 1897, to September, 1901, he was employed as telegraph operator, cashier, and agent for the Baltimore and Ohio Railroad, in Ohio and Indiana, and for the Northern Pacific Railway, in Washington.

In September, 1901, he entered Purdue University, from which he was graduated with the degree of Bachelor of Science in Civil Engineering in 1904. In June, 1909, he received the degree of Civil Engineer from the same University.

From August, 1904, to July, 1905, he was employed as Rodman, Field Draftsman, and Transitman, respectively, on the Galena and Wisconsin Divisions of the Chicago and Northwestern Railway, and from July to November, 1905, he served as Transitman, and Assistant Engineer in charge of 18 miles of construction, on the Wyoming and Northwestern Railway. In December, 1905, he resigned his position with the Wyoming and Northwestern Railway Company to go to Seattle, Wash., where he was engaged, as Field Engineer for the Seattle Engineering Company, on various projects and surveys for hydro-electric developments, and in February, 1907, he was made a member of the firm and Vice-President of the Company.

In October, 1908, Mr. Cook's health failed, and he was forced to abandon his Seattle practice and move to Phœnix, Ariz., where he again engaged in general engineering work. He organized the Phœnix Engineering Company, of which he was President, and built a number of structures. Later, he made a specialty of road construction, in which branch of the profession he attained marked success and worked up quite an extensive consulting practice.

In 1906, he was married to Cora May Caulkins, of Lafayette, Ind., who, with one daughter, survives him.

Mr. Cook was a man of fine character, and his opinions and conclusions were highly respected by other members of the Profession. He had an exceptionally pleasing personality and was loved by all who knew him.

While at college, he took an active part in athletics, making a good record in the pole vault. He belonged to the Tau Beta Pi Fraternity and to the Order of Masons, and was also a member of the Pacific Northwest Society of Engineers.

Mr. Cook was elected an Associate Member of the American Society of Civil Engineers, on May 3d, 1910.

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* Memoir prepared by O. P. M. Goss, Assoc. Am. Soc. C. E.

SAINT GEORGE HENRY COOKE, Assoc. M. Am. Soc. C. E.*

Died January 12th, 1915.

Saint George Henry Cooke was born in Philadelphia, Pa., on June 2d, 1883. He was the only son of Dr. George Henry Cooke, Rear-Admiral, U. S. N. (*Retired*), and Sarah Lyon Cooke. Mr. Cooke's father is of German descent, and his mother, who died in February, 1905, was a daughter of the Rev. Augustus Inloes Lyon, a prominent clergyman of Virginia, in his day.

Mr. Cooke attended the Hamilton School, a private preparatory school, in West Philadelphia, and in the fall of 1898 entered the Pennsylvania Military College, at Chester, Pa., from which he was graduated in June, 1902, with the degree of C. E.

Immediately following his graduation, he entered the employ of Roydhouse, Arey and Company, and, from 1902 until 1905, was in charge of engineering work on factory buildings, storage warehouses, a round-house for the Baldwin Locomotive Works, and other large operations. He resigned his position as Chief Engineer with this firm in December, 1905, to follow railroad engineering. From December, 1905, until August, 1906, Mr. Cooke was Transitman and Masonry Inspector for the Tidewater Railway Company, of Virginia (now the Virginian Railway Company), and, from August, 1906, until September, 1907, he was Resident Engineer for the same Company. From December, 1907, until April, 1909, he was engaged in general engineering in Chester, Pa., and for one year of this time served as Borough Engineer for Ridley Park, Pa.

In 1909, Mr. Cooke opened an office in Philadelphia for general engineering practice, and from October, 1911, until July, 1912, he was employed as Inspector, for the City of Philadelphia, of the buildings being erected for the Home for the Indigent at Holmesburg. From July, 1912, until his death in January, 1915, he was unable, owing to ill health, to engage in any form of business.

In the fall of 1908, Mr. Cooke, who was an enthusiastic military engineer, became quite active in the formation of a militia engineer company to be recruited from the members of the Engineers' Club of Philadelphia, and, on January 8th, 1909, he was elected Captain of Company B, Engineer Battalion, N. G. P., the new militia company thus formed. He served in this capacity until May, 1913, when, owing to ill health, it became necessary for him to resign his commission. As its leader, he brought this organization to a very high standard of proficiency, excelled by none in the State of Pennsylvania. For 10 months previous to his resignation, he had been endeavoring to

* Memoir prepared by James D. Faires, Esq., Philadelphia, Pa.

regain his health in order to take up again and continue his duties with this company, but found that it would be impossible for him to do any further active work in this line.

In the fall of 1914, Mr. Cooke went to Fort Bayard, N. Mex., in a further effort to regain his health, but it was denied him, and he died there on January 12th, 1915.

Mr. Cooke was married in 1906 to Miss Isabel A. Dalmas, daughter of Mr. and Mrs. Louis Dalmas, of Glenolden, Pa., who, with three children, survives him.

He was an active church worker, being a member of Christ Episcopal Church, of Ridley Park, of which he was also a Vestryman. He was a Member and a Director of the Engineers' Club of Philadelphia, and a Member of the Loyal Legion.

Mr. Cooke was elected an Associate Member of the American Society of Civil Engineers, on January 5th, 1909.

WILLIAM GOMER DAVIES, Assoc. M. Am. Soc. C. E.*

Died May 9th, 1915.

William Gomer Davies was born on October 26th, 1877, at Doylestown, Wis. His paternal grandfather, David Davies, was a woolen manufacturer, and several times Mayor of Llanidloes, Wales. His son, Edward Gomer Davies, came to America when a boy of nineteen, arriving on April 14th, 1865. For several years prior to the birth of William (his fourth child), he was studying medicine, and received his degree afterward. Mr. Davies' maternal grandfather was an English Army Officer, who won a medal for bravery in the Battle of Alma, during the Crimean War, and died from wounds received at that time.

Mr. Davies was graduated from the De Smet, S. Dak., High School, in 1893. He attended the Iowa State College of Agriculture and Mechanic Arts in 1895, and Pomona College, California, from September, 1896, to 1898. He entered the Engineering Department of the University of California in 1900, receiving the degree of B.S. in 1903. He completed his college work by taking a post-graduate course at Cornell University in 1905-06.

The courage and industry possessed by his parents and grandparents were passed on to Mr. Davies, whose ability as a student and worker in his chosen field are membered well by his friends and associates. From early boyhood he was thrown on his own resources, working his way through college and university, and doing well, and with all his might, whatever his hand might find to do, whether it was teaching school in Kingsbury County, South Dakota, and at San Bernardino, Cal., or doing odd jobs at the University of California, the experience gained at that time giving him confidence to meet the graver emergencies of later life.

While attending the University of California, Mr. Davies spent one or two vacations in the work of stream gauging. From May, 1903, to April, 1909, he was employed in the United States Reclamation Service on the Minidoka and Boise Projects in Idaho. He served, during the first year, as Hydrographer in Eastern Idaho and Western Wyoming, gauging streams and canals and making a reconnaissance on the Upper Snake River and its tributaries in Idaho, for the purpose of determining the irrigation possibilities of that region. Clearness of vision, a conscientious regard for detail, and breadth of view combined with great industry, especially fitted him for research work of this kind. It was because of these qualities that Mr. Davies, early in his professional career, established a reputation for thorough-

* Memoir prepared by D. W. Ross, M. Am. Soc. C. E.

ness. He possessed an analytical mind, and his reports were always full, clear, and direct, even extraneous matter being treated as a by-product of direct effort to be carefully classified and arranged for use at a future time.

While in the United States Reclamation Service, Mr. Davies also had charge of the location and construction of much of the distributing system of canals on both the Minidoka and Boise Projects, being in immediate charge of a large amount of construction work done by the landowners under a co-operative plan.

In April, 1909, Mr. Davies resigned from the Reclamation Service to accept the position of Principal Assistant Engineer on the irrigation project of the Sacramento Valley Irrigation Company, in the Sacramento Valley, California, which position he held until August 19th, 1913. During this period he had immediate supervision of extensive field operations, involving the making of a topographic survey of about 100 000 acres of land with 1-ft. contour intervals, the location and construction of more than 800 miles of irrigating canals, laterals, and drains, with about 4 000 structures of various kinds, in addition to more than 200 miles of roads, costing in all about $2 500 000. All this work was executed by him with the conscientious regard for detail which always characterized his efforts.

From August, 1913, to January, 1914, Mr. Davies was Resident Engineer on the construction of the Clear Lake Dam, in Lake County, California; and from January, 1914, until his death, he was employed by the State Reclamation Board of California, collecting and checking flood data and classifying lands in connection with the proposed flood-control system of the Sacramento and San Joaquin Rivers. This work involved the studying of engineers' reports, County Assessors' classifications and assessment rolls of swamp land, drainage, reclamation, and protection districts, the location of high-water marks and flood areas, covering in all an area of more than 1 700 000 acres, and an examination of the records of more than twenty counties. It was through Mr. Davies' untiring work and sound judgment that the Board of Assessors was enabled to determine so readily the rate of assessments to cover the great work contemplated for this district.

Although Mr. Davies went about his work with all his might, his mind was not always dwelling on engineering subjects. In fact, he regarded the work on which he was engaged, not as an end in itself, but as a means to the end on which the best qualities of his mind and heart were frequently brought to bear. He was a dreamer as well as a worker, and his dreams were of a world made better and brighter when men become wise enough and fair-minded enough to share with the less fortunate the advantages which have followed the intelligent control of Nature's laws.

The need of the present hour was his greatest inspiration, and with him prayer found its best expression in action. It was his habit to lend more than mere moral support to movements designed for social betterment. His support of temperance was given in his usual vigorous way, but with such fairness as to command the respect of even his bitterest opponents. His every-day life was characterized by that unostentatious goodness which lies at the foundation of true Christian manhood. His fight for the right was always vigorous, but with consideration and charity for those whose position might obviously have been open to condemnation.

Mr. Davies died of pneumonia on May 9th, 1915, beloved by all who knew him well. In his death the Profession has lost a member of real worth—one who would have reached the very top. His short life was full of useful work, which he heartily enjoyed, but which was regarded by him as a means to the higher purposes of existence, a life guided by high ideals.

He was married in January, 1908, to Louisa Babcock Flanders, of Porterville, Cal., who, with one daughter, Charlotte Helen, survives him. His father, Dr. Edward Gomer Davies, and his mother are now living at Yankton, S. Dak.; his brother and sisters are Dr. David Davies, of Woonsocket, S. Dak., Mrs. W. C. Lusk, of Yankton, S. Dak., Mrs. O. H. Grace, of Akron, Colo., Miss Ruth Davies, of Yankton, S. Dak., and Miss Autumn Davies, of Omaha, Nebr.

Mr. Davies was elected a Junior of the American Society of Civil Engineers on February 2d, 1904, and an Associate Member on November 8th, 1909.

HAROLD HANSEN FITTING, Assoc. M. Am. Soc. C. E.*

DIED JANUARY 7TH, 1915.

Harold Hansen Fitting was born on August 21st, 1885, in Grand Rapids, Mich., and moved to California with his parents in October, 1891. He prepared for college at the High School of San Bernardino, Cal., and entered Stanford University in August, 1904, from which he was graduated, with the degree of A. B. in Civil Engineering, in 1909. Prior to entering the University, he was engaged with the Santa Fé Railway on mechanical work in the shops in San Bernardino and, in the course of his studies, both during vacation periods and during a year's absence from the University, he sought and secured work which gave him practical experience in engineering.

In 1906, Mr. Fitting spent the summer as Chainman and Leveler in the employ of the City Engineer of San Bernardino, Cal., and from May, 1907, to August, 1908, he was Draftsman and, later, Assistant Engineer for the Oregon Railroad and Navigation Company on the revaluation of its lines and in charge of parties reporting on the physical condition and preparing estimates for one-half of its mileage.

After his graduation, Mr. Fitting was employed for about one year (1909-10) by the Southern Pacific Company in compiling estimates of revaluation for that Company's lines in Oregon, after which, in 1910, he took charge of the field office of the Stone and Webster Engineering Corporation on the Rubicon hydro-electric development in California.

From November, 1910, until his death, with the exception of only a very few months, Mr. Fitting was with the engineering firm of Duryea, Haehl and Gilman, first as Assistant Engineer and later as Office Engineer. He was entrusted with important problems of engineering investigation and design in power and irrigation work, notably the works of the South San Joaquin Irrigation District, covering about 71 000 acres in the San Joaquin Valley, California, and, during the last year, investigations in connection with the large hydro-electric power project under construction for the Mexican Northern Power Company on the Rio Conchos, State of Chihuahua, Mexico. At the time of his death, he was on temporary leave, being engaged on valuation work for the Oakland, Antioch and Eastern Railway.

On September 20th, 1911, Mr. Fitting was married to Miss Della C. Barnhart who, with a son, survives him. He also leaves a father and two brothers, both the latter being engineers.

On January 6th, 1915, Mr. Fitting was injured in an elevator accident in San Francisco, and died on the day following without

* Memoir prepared by Edwin Duryea, Jr., M. Am. Soc. C. E., and H. L. Haehl, Assoc. M. Am. Soc. C. E., a Committee of the San Francisco Association of Members of the American Society of Civil Engineers.

regaining consciousness. The accident resulted in the death of one other passenger and the serious injury of nine more, three of whom were engineers engaged on the valuation work with him.

When Mr. Fitting began his work with Duryea, Haehl and Gilman in 1910, his extraordinary worth and ability was quickly appreciated by the writers, and that appreciation steadily grew. He was a constant inspiration to his associates, not only because of his extreme energy and application to his work, but because he radiated an enthusiasm and good feeling which was felt by all about him.

Mr. Fitting had the ability to get good from his every activity, and was able to apply his knowledge and experience to every situation. Coupled with a fine mind which found real enjoyment in his work, he was an example of the clean-thinking, straightforward man to whom success in his Profession and high standing in the community are assured. Had he lived, he could not have failed to make his impress on the Engineering Profession at large, and his death was a distinct loss, not only to his many friends and associates, but to his Profession as a whole.

Mr. Fitting was elected a Junior of the American Society of Civil Engineers on May 2d, 1911, and an Associate Member on September 3d, 1913. He was also a member of the San Francisco Association of Members of the American Society of Civil Engineers.

LEON LINCOLN GAY, Assoc. M. Am. Soc. C. E.*

Died May 6th, 1914.

Leon Lincoln Gay, the only son of Carlos E. and Calista Gay, was born at Barton Landing, Vt., on May 30th, 1879. He obtained his education at the public schools, and was a graduate of St. Johnsbury Academy. He then entered the Sheffield Scientific School, of Yale University, from which he was graduated in 1901, with the degree of Ph.B. While at Yale, he was a member of the Track Team and made some good records as a long-distance runner.

In the fall of 1901, Mr. Gay went to Idaho to become Assistant Engineer in charge of the construction of a small power plant at Horseshoe Bend, under A. J. Wiley, M. Am. Soc. C. E., Consulting Engineer. On the completion of this plant, he entered the United States Reclamation Service as Engineering Aid, and was engaged on the preliminary work for the Minidoka Project, in Idaho, during the greater part of 1903 and 1904. During the construction of the Minidoka Dam, on Snake River, he was Assistant Engineer on that work. On its completion, he was assigned to take charge of the building of the original Jackson Lake Dam, near Yellowstone Park, in Wyoming, and made an enviable record in completing this temporary dam at a remarkably low cost. Later, he was Principal Assistant Engineer in charge of the construction of the Boise River Dam, of the Reclamation Service, near Boise, Idaho.

In 1908, Mr. Gay resigned from the Reclamation Service to accept a position as Engineer for a large irrigation enterprise in the Sacramento Valley, California. The following year he was attacked with tuberculosis, and the last five years of his life was a continuous fight with this dread disease to which he finally succumbed while undergoing an operation in Montreal, Que., Canada.

Mr. Gay was known by the large number of engineers of his acquaintance in the West to be a man of high moral character and absolute integrity. He was considered one of the most capable young engineers in the line of irrigation in the United States, and previous to his unfortunate illness he had executed some important and successful work. His misfortune and death are peculiarly sad. He was a typical New Englander, and among his near friends his humor and originality gained for him the affectionate nickname of "the Yank".

He is survived by his father and stepmother, of Orleans, Vt., as well as by his two sisters, Mrs. Wallace C. Gilpin, of Barton, Vt., and Mrs. Fred E. Parker, of Falls Church, Va.

Mr. Gay was elected a Junior of the American Society of Civil Engineers on December 5th, 1905, and an Associate Member on October 2d, 1907.

*Memoir prepared by C. W. Joslyn, Esq., and F. C. Horn, M. Am. Soc. C. E.

LOUIS THOMAS FRANKLIN HICKEY, Assoc. M. Am. Soc. C. E.*

Died May 18th, 1915.

Louis Thomas Franklin Hickey was born in San Rafael, Cal., on May 18th, 1886. He received his preparatory education in the public schools of that place, and was graduated, in May, 1907, from the College of Civil Engineering, University of California, with the degree of B.S. During the college vacations, he was connected with various engineering enterprises, and, on graduation, became Assistant Engineer in the Geological Department of the Southern Pacific Railroad, being detailed on engineering work for the Kern Trading and Oil Company.

Mr. Hickey left the service of the Southern Pacific Company in the fall of 1909 and, from that time to December, 1913, was engaged in civil and hydraulic engineering work in the Pacific Coast States and Mexico. Particularly notable was his work in connection with the investigation of the proposed San Francisco water supply, in which an extensive hydrographic study of the Alameda Creek water-shed and the Hetch Hetchy drainage basin was made.

Since December, 1913, he had devoted his time to private practice in San Francisco, where he had made an enviable reputation for himself among those who knew of his work.

In mourning the loss of Mr. Hickey, it is with the feeling that the Engineering Profession has been deprived of one of its most promising members, and a large circle of friends a singularly congenial spirit.

On October 5th, 1912, he was married to Ethel Anita Jackson, of San Francisco, Cal., who survives him.

Mr. Hickey was elected an Associate Member of the American Society of Civil Engineers on June 24th, 1914. He was also a member of the San Francisco Association of Members of the Society.

* Memoir prepared by William Reinhardt, Assoc. M. Am. Soc. C. E.

ALBERT LLOYD HOPKINS, Assoc. M. Am. Soc. C. E.*

Died May 7th, 1915.

Albert Lloyd Hopkins, the son of Stephen DeForest and Elizabeth G. Hopkins, was born on September 7th, 1871, at Glens Falls, N. Y. His early life was passed in that city and in Troy, N. Y., and there his early education was received.

In 1888, he entered Rensselaer Polytechnic Institute for the course in Civil Engineering. He was graduated in 1892, with high honors, and, in later years, the Director of the Institute said of him: "About ten per cent. of those who apply are admitted to the Institute, about twenty per cent. of those admitted are graduated, and among these, once in a while, we find a Hopkins."

After a few months' work in an architect's office in Chicago, in the summer of 1892, Mr. Hopkins was appointed to a position in the Bureau of Construction and Repair, of the Navy Department, at Washington, D. C., which position he held for about 18 months, being sent to Newport News, Va., in February, 1894, as a member of the staff of Naval Constructor J. J. Woodward, U. S. N., the first Superintending Constructor assigned to duty at the works of the Newport News Shipbuilding and Dry Dock Company.

In the summer of 1897, Mr. Hopkins was transferred from Naval Constructor Woodward's office and assigned to the Graduate School of Naval Architecture at the U. S. Naval Academy, Annapolis, Md., at which place he was an Instructor and Lecturer on naval architecture and ship construction.

At the outbreak of the Spanish War, in April, 1898, the officers and students of the Academy joined the fleet, and Mr. Hopkins was assigned to the Naval Station at Key West, Fla., the nearest station to the blockading fleet operating in Cuban waters. While at Key West, he was in charge of all the construction and repair work done for the fleet at that station. He was also active in improving the efficiency of the plant, installing much new equipment and rendering it capable of serving as a repair station for naval purposes.

It was while engaged in the work at Key West that Mr. Hopkins received from the late W. A. Post, M. Am. Soc. C. E., then General Superintendent of the shipyard, an offer to return to Newport News and enter the service of the Newport News Shipbuilding and Dry Dock Company. He accepted, and, in August, 1898, returned to Newport News as the personal assistant of Mr. Post.

The able constructive work done by Mr. Hopkins in the years following will always be remembered by those who were associated with

*Memoir prepared by the Newport News Shipbuilding and Dry Dock Company, New York City.

him. Gifted with a rarely keen mind, he was quick to grasp the essentials of any situation. His education and training qualified him to choose unerringly the right course to pursue, and his strong will and personality enabled him to carry to its logical conclusion the course so chosen. United with these strong characteristics were an unfailing courtesy and a consideration for others, which endeared him to all his associates.

In 1905, when Mr. Post was made General Manager of the Company, Mr. Hopkins was appointed Assistant General Manager, and, in 1911, when Mr. Post succeeded to the Presidency, on account of the death of Mr. C. B. Orcutt, Mr. Hopkins was made Manager.

On the death of Mr. Post in February, 1912, Mr. Hopkins was elected Vice-President, and became the chief executive officer of the Company, with headquarters in New York City. In this new field, he fully sustained the high position accorded to his predecessors, and it was quickly and widely recognized that an able executive was directing the Company's affairs. In his own Profession, and also among the men whom, because of his Profession, he was called on to meet—bankers, lawyers, and men of large business affairs—his influence was deep and ever increasing. His election as President of the Company, in March, 1914, was recognized as a well-earned tribute to his ability and his devotion to the interests committed to his care.

Loyal to his friends and associates, loyal to those who trusted their interests to him, loyal always to his own high ideals, well and truly does he deserve the tribute expressed in the following telegram sent by Mr. Huntington to the Company:

"Mrs. Huntington and I are distressed beyond expression at the death of Mr. Hopkins. We believe the Company and all the employees share our sorrow and will mourn the loss of a splendid officer and a noble man."

To those who knew him best, to his friends and associates of many years, the tidings of the *Lusitania* disaster brought a sure message—well they knew that the gentle, kindly courtesy, the ever-present, instinctive disposition to serve others before considering himself, would unfailingly include him among those who died that others might live.

In June, 1906, Mr. Hopkins was married to Miss May Davies, of Chase City, Va., who, with their daughter, May Davies Hopkins, survives him. He is also survived by his mother, of Glens Falls, N. Y., and his brother, Charles E. Hopkins, of Hudson, N. Y.

He was a member of the American Society of Naval Engineers, the Society of Naval Architects and Marine Engineers, and the American Academy of Political and Social Science.

Mr. Hopkins was elected a Junior of the American Society of Civil Engineers on April 3d, 1894, and an Associate Member on April 3d, 1901.

JESSE SIDWELL MATSON, Assoc. M. Am. Soc. C. E.*

Died July 4th, 1914.

Jesse Sidwell Matson was born at Pennsville, Ohio, on November 19th, 1862. He was educated in the public schools of that place and at Marietta College, Marietta, Ohio.

From 1888 to 1895, Mr. Matson was engaged as a teacher of mathematics in the Preparatory Department of Marietta College and in the Marietta Public Schools. In 1895, he went to Malta, Ohio, where he remained as a teacher of mathematics in the High School, until 1899.

During 1899 and 1900, Mr. Matson served as Transitman on construction work for the Pennsylvania Railroad, and, from 1901 to 1902, he was employed by the County Engineer of Ashtabula County, Ohio, on road work, sewers, preliminary surveys for trolley lines, etc. He also served as Superintendent of the Public Schools at Windsor, Ohio.

In 1902, Mr. Matson was elected County Engineer of Ashtabula County, which position he held for ten consecutive years, serving with great credit to himself as well as to the best interests of the County.

While engaged on road construction at Mesopotamia, Ohio, Mr. Matson was stricken with apoplexy and died on July 4th, 1914, after an illness of five days.

Mr. Matson had had the advantage of being in office when the new era in road building began. He became thoroughly conversant with that branch of engineering, and was held to be one of the most capable road engineers in Northeastern Ohio. He was also extensively engaged in bridge design and construction, and his plans were widely used. At the time of his death, he had almost completed a set of tax maps for Ashtabula County, which have been pronounced by competent judges to be the best of the kind in the State.

A giant physically, Mr. Matson was a quiet, most lovable character, and his life was one of complete devotion to the well-being of his parents and family. He was always genial, courteous and unassuming. He was honorable in all his dealings, and no man was better known or more highly respected in Ashtabula County. His sudden death was a great shock and a distinct loss to the entire community.

Mr. Matson had well-defined literary taste and ability, and although constantly engaged in his professional work, he was always increasing his library with modern scientific and religious works. After an absence from home, he keenly enjoyed examining the new books acquired since his last visit. He was religiously inclined, and, from

* Memoir prepared by the Secretary from information furnished by Mrs. J. S. Matson, supplemented by material on file at the Society House.

his early manhood, had been a faithful and consistent member of the Methodist Episcopal Church.

While at Windsor, Ohio, Mr. Matson was married to Miss Grace Barnard, who, with a son and a daughter, survives him. He is also survived by a sister, Mrs. L. B. Simpson, of McConnellsville, Ohio, and a brother, Mr. A. H. Matson, of Malta, Ohio.

He was a member of the Western Reserve Lodge of Odd Fellows, the Masonic Blue Lodge, of Ashtabula, Ohio, and a Charter Member of the Young Men's Christian Association.

Mr. Matson was elected an Associate Member of the American Society of Civil Engineers on July 1st, 1908.

RALPH ASHUR PIKE, Assoc. M. Am. Soc. C. E.*

DIED MAY 13TH, 1915.

Ralph Ashur Pike was born in East Woodstock, Conn., on June 3d, 1879, where he spent the early years of his life with his parents. He was prepared for college at the Woodstock Academy, and entered Sheffield Scientific School, Yale University, in September, 1898, from which he was graduated with honors in 1901. During vacation periods, he secured work which gave him practical experience in engineering.

In 1901, Mr. Pike spent the summer in the employ of the New York Central and Hudson River Railroad Company, as Chainman, and, from October, 1901, to June, 1903, he was Rodman and Leveler in charge of some work on bridge and tunnel construction, and was also Chief of Party on a survey for the Pennsylvania Railroad Company.

From June, 1903, to May, 1905, Mr. Pike was employed by the New York, New Haven and Hartford Railroad Company, as Draftsman in the office of the Chief Engineer, in New Haven, Conn., and from May, 1905, to April, 1908, he was with the New York Central and Hudson River Railroad Company, as Draftsman and Assistant Engineer in charge of a squad, designing concrete arches, culverts, retaining walls, substructures for steel bridges, and building foundations, etc., and, later, in the New York office, on various studies and estimates for removing the tracks from the surface of Eleventh Avenue, New York City, by subway or elevated.

In the spring of 1908, Mr. Pike began work for the New York State Public Service Commission, First Division, designing and checking steel and reinforced concrete on various parts of the proposed Tri-Borough Subways and, later, as Designer in charge of a squad making working drawings for the four-track turn-out (with no grade crossings) from the four-track Fourth Avenue Subway to the future Coney Island Extension, the estimated cost of which was $1 000 000.

Mr. Pike was stricken with tuberculosis in January, 1911, and, from then until his death, was unable to work much of the time. In the fall of 1911 he, with his wife, moved to El Paso, Tex., in the hope that the change of climate would help him. In December, 1911, and January, 1912, he was employed by the Mexico and Northwestern Railroad Company, as Draftsman, in Mexico, but was obliged by the revolutionary disturbances to return to the United States.

In May, 1912, Mr. Pike left Texas for Los Angeles, Cal., having been advised to seek a lower altitude; here, he assisted Wilkie Woodard, M. Am. Soc. C. E., in a general civil and landscape engineering practice, until June, 1913, at which time the physicians advised com-

* Memoir prepared by Wilkie Woodard, M. Am. Soc. C. E.

plete retirement and rest, and he entered La Vine Sanitarium, Pasadena, Cal., where he remained until his death.

Mr. Pike was one of those rare combinations of thorough ability, good judgment, and consideration of the rights of others. He was highly appreciated and trusted by all who knew him.

On June 28th, 1904, he was married to Miss Alice L. Prindle, who survives him. He is also survived by his father and mother who still live in East Woodstock, Conn.

Mr. Pike was elected a Junior of the American Society of Civil Engineers on March 6th, 1906, and an Associate Member on January 31st, 1911.

PHILIP MORRIS PRITCHARD, Assoc. M. Am. Soc. C. E.*

DIED JULY 8TH, 1914.

Philip Morris Pritchard was born on April 29th, 1872, at Rock Ferry, near Birkenhead, England. He was educated in London, where his father was in business as a tea importer and merchant. At an early age the boy showed a strong liking for Engineering, and after his graduation from Kings College, London, in 1891, he was sent to Newcastle-on-Tyne to serve an apprenticeship of four years with the firm of R. and W. Hawthorn, Leslie and Company, Limited, Locomotive and Marine Engineers and Shipbuilders, where he received a thorough practical workshop training. During three years of his apprenticeship, he attended the Engineering Course at Durham College of Science, at Newcastle, and, on his graduation, was awarded an Associateship in Science.

In June, 1895, Mr. Pritchard was engaged as Draftsman and Assistant Engineer at the Tennant-on-Tyne Works of the United Alkali Company, Limited. In January, 1896, the Company organized its Engineering Department at Widnes, England, and he was brought from the Tennant Works to occupy an important and responsible position on the engineering staff at the latter place. This promotion eventually led to his appointment as Chief Engineer of the Company, a position which he held for about 12 years and at the time of his death on July 8th, 1914.

While in charge of the Engineering Department of the United Alkali Company, Limited, Mr. Pritchard was engaged in the design and erection of much new plant in the English and American works of the Company and did valuable work in its Spanish mines.

All through his career, Mr. Pritchard showed brilliant ability as an Engineer, and his death is a great loss to the Company as he was greatly esteemed both by the Directors and his colleagues.

He was a Member of the Institute of Mechanical Engineers and of the Society of Chemical Industry. He was also an Associate Member of the Institution of Civil Engineers.

Mr. Pritchard was elected an Associate Member of the American Society of Civil Engineers, on June 6th, 1900.

* Memoir prepared by the Secretary from information supplied by the United Alkali Company, Limited, Widnes, England, supplemented by information on file at the Society House.

CHARLES EZEKIEL RASINSKY, Assoc. M. Am. Soc. C. E.*

DIED DECEMBER 26TH, 1914.

Charles Ezekiel Rasinsky was born at Mohileff, Russia, on March 24th, 1865. His early life was spent in his native town, and there he acquired his primary education and was graduated from the local high school. His parents were well to do, and he was desirous of attending the University, but was unable to do so on account of the restrictions placed on the number of Jewish students admitted. Partly on this account and partly because of his liberal views, he decided to come to America, which he did in 1883 without his parents' consent. He supported himself in several lines of work for three years at various places; coming to Cincinnati, Ohio, in 1886, he entered the University for the Civil Engineering course. He worked his way through college and was graduated in 1890 with distinction. Both during and after his college career, he was a profound student of technical literature in English, French, German, and Russian.

After his graduation Mr. Rasinsky entered the Engineering Department of Cincinnati as a Rodman. He remained in this position only a few months, becoming, in the fall and winter of 1890-91, Inspector on the filtration plant at Jeffersonville, Ind., under John W. Hill, M. Am. Soc. C. E., Chief Engineer. Early in 1891, he was appointed Assistant Engineer in the Engineering Department of Cincinnati, where he remained until 1897, his work consisting largely of street and highway construction.

During 1898 and 1899, Mr. Rasinsky was Assistant on the construction of the new Cincinnati Water-Works under the late L. G. F. Bouscaren, M. Am. Soc. C. E., Chief Engineer, being engaged especially in the detailed calculations for reservoir and heavy masonry layout, and also in the design of structural and special casting work.

In 1900, he again entered the City Engineering Department of Cincinnati and continued there until 1906. His work during this time consisted in street construction and layout, and the design and construction of retaining walls, bridge piers, and abutments. During all these years Mr. Rasinsky was doing a great deal of structural designing and detailing for local shops outside of regular office hours, and, in 1906, he entered the employ of L. Schreiber and Sons Company, as a Structural Designer and Detailer. After a few months, however, he went with the Ohio Electric Company, at Dayton, Ohio, as Bridge Engineer on the proposed reconstruction of its line from Dayton to Hamilton, Ohio. He designed all the structural and reinforced con-

* Memoir prepared by H. F. Shipley, M. Am. Soc. C. E.

crete work on this line, some of which, however, was not put into use for several years on account of the curtailment brought on by the financial stringency of 1907.

In 1908, he again entered the service of the City Engineering Department of Cincinnati, this time in the Sewerage Division, under J. H. Sundmaker, City Engineer. In 1910-11, when the writer became City Engineer, Mr. Rasinsky assumed general charge of the entire Sewer Division. This branch of the Engineering Department had been retrograding for many years, and had become an affair of mere routine, no work of a general nature being carried on. Mr. Rasinsky immediately began a reorganization along more advanced and modern lines, but lack of financial support prevented the carrying out of his plans on the broad scale required to achieve real results in Cincinnati.

In 1912, he entered private practice as a Civil and Consulting Engineer, and continued as such until his death. During the last two years of this time he was associated with the writer under the firm name of Shipley and Rasinsky.

Mr. Rasinsky was married in 1891 to Miss Rebecca Tuttleman, who, with three children, survives him.

He was a man of sterling character and of very considerable scientific attainments, especially in the higher mathematics. He was a pioneer local designer in reinforced concrete, and was versed in all the literature of this subject in four languages. He never reached the higher ranks in the engineering world, but he was a man who matured late in life, and was apparently ready for his best work at the time of his sudden death.

Mr. Rasinsky was elected an Associate Member of the American Society of Civil Engineers on July 9th, 1912.

WILLIAM DE HERTBURNE WASHINGTON, Assoc. M. Am. Soc. C. E.*

DIED AUGUST 30TH, 1914.

William de Hertburne Washington was born at Clover Sea, Hanover County, Virginia, on June 29th, 1863. He was the son of Lewis W. and Ella Basset Washington, and was a descendant of Charles, a brother of George Washington.

Mr. Washington was educated at Mary Schools and spent a term at college in special technical and mathematical study. He was to have entered the United States Navy, but no vacancy occurring at the time, he secured employment, on March 1st, 1880, on a preliminary survey for The Coal and Iron Railroad, in West Virginia, attaining the position of Instrumentman. The following year he was engaged as Assistant Engineer on the West Virginia Central and Pittsburgh Railway, and spent another year or more getting out timber and prospecting for ores and minerals on the lines of the Norfolk and Western and Chesapeake and Ohio Railways.

Mr. Washington then came to New York City and was employed for some time in the office of the Chief Engineer of the American Atlantic and Pacific Ship Canal Company (the original Nicaragua Canal Company), and in other mechanical engineering work. In October, 1885, he was appointed United States Consul at London, Ont., Canada, and, in addition to his official duties, was allowed to act as representative of the firms of Messrs. Andrew Brothers, and Ferris and Walladay, Water-works Engineers and Contractors, reporting on and negotiating for the construction of water-works or hydraulic plants at Toronto, St. Thomas, Chatham, etc.

On his return to New York City, Mr. Washington organized the Hydraulic Contracting Company which built the water-works at Conneaut, Ohio, and hydraulic plants at many other places. When the firm was dissolved, he organized and became President and General Manager of the Hydraulic Construction Company which designed and constructed forty very important industrial plants in the United States.

After remaining in the construction business for about ten years, Mr. Washington went into the development of real estate, and at the time of his death, was President of the New Center Company, the Eastern Warehouse and Realty Company, etc.

In 1913, he was appointed Consulting Engineer to the Highway Commission of the State of New York, by Governor Sulzer, and was a Delegate to the Third International Road Congress, held that year in London, England. After the adjournment of the Congress, he visited various European countries for the purpose of inspecting roads

* Memoir prepared by Ernest Abs-Hagen, Esq., supplemented by information on file at the Society House.

and pavements, and on his return to the United States, made an elaborate report on his observations to the State Highway Commission.

Mr. Washington had recently published a book entitled "Progress and Prosperity," which embodied a comprehensive study of the mediums of development by the influence of railroad and highway transportation. He was also a contributor to the discussion on "Road Construction and Maintenance" before this Society in January, 1914.

He was the organizer and President of the West Virginia State Society, a member of the Sons of the Revolution, the Southern Society, the Virginia Historical Society, the American Society of Mechanical Engineers, the American Institute of Mining Engineers, and a Fellow of the National Geographical Society. He was also a member of, and until a short time before his death made his home at, the Calumet Club, in New York City.

Mr. Washington had just returned to New York City after a trip to Canada when he was stricken with cerebro-spinal meningitis, and died August 30th, 1914, after a short illness. His body was taken to Charlestown, W. Va., and interred in the family cemetery there.

He was unmarried, and is survived by a nephew and two sisters.

Mr. Washington was elected an Associate Member of the American Society of Civil Engineers on October 5th, 1892.

FRANK BURNS STOREY, Jun. Am. Soc. C. E.*

Died April 21st, 1915.

Frank Burns Storey was born in Rochester, N. Y., on February 23d, 1888. He was the eldest son of Cora Burns Storey and William R. Storey, and the grandson of O. W. Storey who, from 1846 to 1866, served as Resident Engineer and Division Engineer, respectively, on the New York State Canals.

Mr. Storey received his early education in the public schools of Rochester. He was graduated in the Mechanics Arts Course from the Rochester Mechanics Institute, and, for a year, attended the University of Rochester. In 1906, he entered Cornell University, his father's Alma Mater, from which he was graduated in 1910 with the degree of C. E.

During his school and college vacations, Mr. Storey was engaged as Rodman and Draftsman on work for his father, thereby gaining considerable engineering and surveying experience. During his course at Cornell, he specialized in hydraulics and, previous to his graduation, passed a Civil Service examination as Junior Engineer in the Water Resources Branch of the United States Geological Survey.

In July, 1910, he received his appointment and was sent to Portland, Ore., where he remained until May, 1913. During the summers of 1910 and 1911, Mr. Storey was engaged in the field collecting hydrometric data on canals and streams in the Yakima Valley, the winter seasons being devoted to office analyses of such data at the District headquarters at Portland.

In the spring and summer of 1912, he was detailed to make stream measurements throughout the States of Washington and Northern Idaho, and during the fall and winter of the same year, he collected the data for the third† of a series of seven reports on "Water Powers of the Cascade Range", being prepared and published by the U. S. Geological Survey.

In May, 1913, Mr. Storey was transferred to Tacoma, Wash., and, on the establishment of a new District, in June, 1913, to include the States of Washington and Northern Idaho, was made Principal Field Assistant in Eastern Washington and Northern Idaho, having responsible charge of the field work in that part of the new District.

In April, 1914, he was granted a furlough and returned to Rochester, N. Y., to engage in general engineering practice with his father, continuing in this position until his death on April 21st, 1915. Among other work, he had designed several sewers for the town of Gates, N. Y., which are now under construction.

* Memoir prepared by the Secretary from information supplied by C. L. Crandall, M. Am. Soc. C. E.

† U. S. Geological Survey, Water Supply Paper No. 369.

Mr. Storey was a good student, and took such keen interest in his work that he stood the "Cornell test" of doing it well. Hating shams and pretense, he never posed, but was always a gentleman, and the world is better for his having lived in it. He is survived by his parents, two sisters and a brother.

He was a member of the Rochester Engineering Society, the Cornell Society of Civil Engineers, and the Rochester Cornell Club. Mr. Storey was elected a Junior of the American Society of Civil Engineers on March 5th, 1912.

WILLIAM RICHARDSON BELKNAP, F. Am. Soc. C. E.*

Died June 2d, 1914.

William Richardson Belknap, the son of William Burke and Mary (Richardson) Belknap, was born in Louisville, Ky., on March 28th, 1849. He received his early education in the schools of that city, and was prepared for college at the Louisville High School. He then entered the Sheffield Scientific School of Yale University for the select course, and was graduated in the Class of 1869. While at Sheffield, he was a member of Berzelius, captain of the Undine Boat Club, and, in 1869, a member of the *Lit* Board.

After a year's post-graduate course in biology at Yale, Mr. Belknap returned to Louisville, and for three years worked with his father who was in the hardware business there. Subsequently, he spent another year in study in Germany.

On his return to Louisville, he took over his father's business, which at that time was conducted as W. B. Belknap and Company and consisted of one small store dealing almost exclusively in blacksmiths' supplies. Under his management the business grew into the concern known as the Belknap Hardware and Manufacturing Company, employing about 1 000 men, and having representatives in all parts of the United States. Thus, the narrow blacksmith line was broadened out until a description of the Company's manufactures filled a catalogue containing nearly 100 000 items. The present plant consists of modern warehouses with floor space of more than 24 acres and railroad side tracks accommodating 20 freight cars at a time.

In the spring of 1910, Mr. Belknap retired as President of the Company, after having held that office for nearly 30 years. He continued, however, as Chairman of the Board of Directors until his death on June 2d, 1914, following a general breakdown. In his business career he had always kept before himself and his employees the principles advocated by the efficiency engineers of the present day, namely, the elimination of all waste as well as of all waste movements, and he had built up one of the largest businesses in the South from very small beginnings.

Mr. Belknap was a Trustee of Berea College and was active in the support of the Tuskegee and Lincoln Institutes. He was a staunch supporter of the Kentucky Yale Alumni Association and the Kentucky Scholarship Fund, and was the founder of the William R. Belknap Prizes for excellence in geology and biology in the Sheffield Scientific School.

* Memoir prepared by the Secretary from information supplied by William B. Belknap, Esq.

He was a Director of the Associated Charities of Louisville for nearly 20 years. He was also actively interested in both the Young Men's and Young Women's Christian Associations, and had served as an Elder of the Warren Memorial Presbyterian Church.

Mr. Belknap was for several years a Director of the Louisville Board of Trade, and had served on the Board of Directors of the Southern Exposition. He was one of the founders of the Salmagundi Club of Louisville, a member of the Country Club, and an Honorary Life Member of the Commercial Club, both of that city.

On December 2d, 1874, he was married to Alice Trumbull Silliman, daughter of Professor Benjamin Silliman, Jr., of Yale University. Mrs. Belknap died in November, 1890, and on February 21st, 1894, he was married to Miss Juliet Rathbone Davison, of Louisville, Ky., who, with one son and four daughters, survives him.

Mr. Belknap was elected a Fellow of the American Society of Civil Engineers on May 28th, 1872.

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